JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

VOL. 28 JANUARY, 1936 No. 1

CONTENTS

Rainfall and Stream Flow Conditions During Floods in Central and Southern New York. By Arthur W. Harrington 1
Mechanical Control in Water Works Operation. By A. H. R. Thomas 6
Pioneering in the Water Works Field. By Morgan D. Hayes 22
How One Company Meets its Problems of Meter Reading, Billing and Collecting. By John C. Houston, Jr. 30
Replacing Wood with Cast Iron Pipe under P.W.A. Conditions. By Richard F. Wagner 38
Water Tanks of Reinforced Concrete. By A. L. Hewett .. 43
The Small Town Water System, Its Problems and Perplexities. By M. H. McGuire 50
The Water Filtration Plant of Hamilton, Ontario. By W. L. McFaul 57
Safeguarding Public Water Supplies from Source to Consumer. By Arthur E. Gorman 65
Preparation and Conditioning of Aluminum Sulphate Solution Before Applying to Water. By Carl Leipold 74
The Development of the Bremerton, Washington, Water System. By Cliff C. Casad 97
Domestic Water Supplies of Idaho. By W. V. Leonard .. 106
Economies in the Selection and Operation of Electric Power for Pumping Equipment. By D. P. Minichan .. 108
Diesel Engines in the Water Works Field. By W. S. Russell 119
Design of Earth Fill Dams. By Wayne W. Wyckoff 127
Soil Compaction Control for Rolled Earth Dam Construction. By R. R. Proctor 134
List of Section Papers. Virginia 142
Abstracts 143

All correspondence relating to the publication of papers should be addressed to the editor, Abel Wolman, 2411 North Charles Street, Baltimore, Maryland.

Because of the necessity for rigid economy, no reprints of articles will be furnished to contributors free of charge. Reprints may be purchased at the usual prices.

• • FAMOUS VICTIMS OF WATER BORNE DISEASES

LOST *to the* Elizabethan Stage . . .
Shakespeare, *Poet and Dramatist*

AS APT as the day they were written, Shakespeare's words are wise axioms today. "Honest water," says the great writer, "ne'er left a man i' the mire."* But a water-borne disease—typhoid fever—was the cause of his own death in 1616.

Three centuries ago there was no known protection against the ravages of water-borne disease. Today—we know that an "honest water" is a safe water—made safe by CHLORINATION. And to properly apply chlorination, you need an honest chlorinator,—honestly designed, honestly built,—in short the W & T Visible Vacuum.

Chlorination's cost is so low—less than a penny per person per year—and control so easy and reliable with W & T Visible Vacuum Chlorinators that you cannot afford to be without it.

An honest public health policy is "The Only Safe Water is a Sterilized Water."

*"Timon of Athens."

Ask for technical publications 38, 157 and 158 describing W & T Visible Vacuum Chlorinators

WALLACE & TIERNAN CO., Inc.

Manufacturers of Chlorine and Ammonia Control Apparatus

NEWARK, NEW JERSEY Branches in Principal Cities. Main Factory Belleville, N. J.

SA-64

JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

The Association is not responsible, as a body, for the facts and opinions advanced in any of the papers or discussions published in its proceedings
Discussion of all papers is invited

Vol. 28 JANUARY, 1936 No. 1

RAINFALL AND STREAM FLOW CONDITIONS DURING FLOODS IN CENTRAL AND SOUTHERN NEW YORK

By Arthur W. Harrington

(District Engineer, U. S. Geological Survey, Albany, N. Y.)

During the afternoon of July 7 and the morning of July 8, 1935 a series of thunderstorms of great intensity centered in an area of some 7,000 square miles in ten counties in central and southern New York. Abnormally heavy rainfall occurred at about the same time in parts of the Mohawk Valley, in the Hudson River valley north of Albany and in other more or less isolated sections, such as the Chaumont River region in Jefferson County. While under ordinary circumstances the flood conditions in these outside areas would have been considered notable, they were so far surpassed by the conditions in the main area that no further reference will be made to them in this paper.

Now that this flood is a matter of history we can examine it in relation to other past floods in the northeastern section of the United States. By almost any standard of comparison the July flood is entitled to rank as a classic. The well-known New England flood of 1927 is the most recent and in fact about the only well-authenticated flood which is at all comparable. In the New England flood the precipitation was in general much less both in total amount and intensity than in the New York flood. The condition of the ground and the mountainous character of the terrain were important factors in causing the high runoff and resultant property damage in New

England in November, 1927. In the July flood the great intensity of the rainfall and the hilly territory made conditions right for an intense runoff. The two floods were therefore similar in runoff characteristics.

The importance of a systematic study of hydrology has been fully appreciated only recently. The intimate relation of this science to the problems of the hydraulic engineer is now quite generally conceded, and it is certain that more attention is being given to continuous records of precipitation and stream flow, as well as to abnormalities in rainfall and runoff than ever before. We can theorize in the realm of probability, but after all the best guide for the future will continue to be the record of the past. Unfortunately for the hydrologist, both records of precipitation and of stream flow are of too short duration to provide much material for a study of cycles, if such exist, nor can they give much more than a rough approximation in the matter of extremes, both high and low. The Geological Survey, which is the chief governmental agency in the collection of stream flow data, has few authentic records extending back more than 35 years. The precipitation records of the Weather Bureau are of somewhat greater length, but still far too short.

In view of these facts, any statement regarding the magnitude of the July flood must be relative only. We do know that this flood far exceeded any maxima heretofore recorded at our gaging stations in the area. For example, at the station on Chenango River near Chenango Forks, a peak discharge of 82,800 second-feet was indicated on July 8. The previous maximum for 26 years was 35,500 second-feet. The testimony of old residents indicates that a flood of comparable magnitude occurred in at least parts of this area in March, 1865. Perhaps then we might think of the July deluge as a "seventy year flood," which of course does not mean that seventy years must elapse before the occurrence is repeated.

PRECIPITATION

A study of all available precipitation records from Weather Bureau stations and elsewhere shows extreme variations throughout the flood area. Binghamton, for example, which suffered severe damage from Chenango River, received about an inch of rain during the period of the principal series of storms. At Hector, north of Watkins Glen, the rainfall totaled 14.23 inches in 47 hours; at Ithaca, 8.12 inches in 36 hours and 5.46 inches during the last 12 hours of the period. Cortland reported a total of 11.54 inches for the storm and 7.67 inches in

24 hours. Delhi, on the West Branch of Delaware River, and on almost the extreme eastern edge of the flood area recorded a 24-hour maximum of 8.52 inches, while a few miles farther east the rainfall was not unusual. Unofficial records, such as measurements by farmers and others of catches of rainfall in exposed receptacles furnish further reliable evidence that 24-hour totals of from 8 to 10 inches were common in many parts of the area. It should be borne in mind when considering these figures that in this region a rainfall of 2 inches or more in 24 hours would normally be considered heavy precipitation. It seems quite probable that the heaviest and most intense rainfall was not recorded at all, either by reason of its occurrence in remote and sparsely populated areas or on account of the fact that any potential unofficial observers were too busy protecting their own lives and property to concern themselves with the collection of rainfall data. The opinion is ventured that in some small areas the series of storms produced 20 inches or more of precipitation with a 24-hour rainfall in excess of 12 inches. These figures seem almost incredible for New York State, but serve to show that the unexpected may happen at any time.

The immediate result of this heavy downpour was to quickly convert the small streams, particularly those with drainage areas of 25 square miles or less, into raging torrents and it was these small streams which caused some of the most spectacular damage. People were drowned. Buildings were wrecked and demolished. Domestic water supplies were polluted and seriously impaired. Large areas of farm land, as for example in the Chenango valley, were covered deeply with gravel, boulders and drift of all kinds by overflowing streams which would normally carry less than one second-foot of water during the summer months. In other cases the effect was reversed and the top soil was deeply eroded or entirely carried away. At Watkins Glen a railroad bridge pier in the center of the gorge provided an obstruction back of which flood debris built up to a great height and which was suddenly released when the pier failed. At Hammondsport, Glen Brook with a drainage area of only 5 square miles quickly cut through a diversion dam which had for many years conducted the stream into a sidehill channel around the town. A large warehouse, built in the stream channel above the dam was quickly demolished and its contents, 1200 barrels of grape brandy were added to the wreckage, as the stream resumed its original course to Keuka Lake. In many other villages located on the banks of

small streams the flood damage was large on account of poor channel conditions or by reason of encroachments on the natural channels by buildings and other structures.

As might be expected, the larger streams rose more slowly and with some notable exceptions the principal damage caused by the main rivers was through inundation, involving serious damage to crops and the interiors of dwellings. It is not the purpose of this paper to recite the physical damage inflicted on highway and railroad bridges, culverts, embankments, buildings and farm land, all of which was impressive enough to anyone who visited the area immediately after the flood.

FLOOD FLOWS

Bare statistics are not impressive but a few figures may serve to give some idea of the intensity of this flood. A few systematic studies of flood flows in the United States have been made. These studies show very few recorded floods on even the smallest streams with peak discharges of more than 1,000 second-feet per square mile. Our own investigation of the July flood is not yet complete and accurate discharge measurements from flood marks and by the slope method are difficult to obtain. It is believed, however, that many small streams of 10 square miles drainage area or less reached peak discharges of from 1,000 to 2,000 second-feet per square mile, figures which are of record breaking magnitude. Glen Brook at Hammondsport, for example, referred to previously, reached a peak discharge in excess of 1,000 second-feet per square mile. In general, the largest streams, with drainage areas of from 500 to 2,500 square miles, showed peak discharges of from 20 to 60 second-feet per square mile. Smaller streams draining from 100 to 200 square miles reached peak discharges of from 125 to 200 second-feet per square mile, remarkably high figures considering the size of the drainage areas. It seems certain, therefore, that this flood will be referred to for many years to come as one of the greatest recorded floods in this part of the United States.

INADEQUATE GAGING

As is usually the case, in critical times the number of regular gaging stations in the flood area was inadequate, practically all of the stations being located on the larger streams. It is unfortunate that no gaging stations, except one on the edge of the flood area, were in operation

on the smaller streams, as the information on the distribution of runoff which such stations would have provided would now be of great value. The stations which were in operation were mostly of the modern automatic type and in nearly every case complete gage-height graphs were recorded throughout the flood period, thus giving information not only on discharge but also on the time of flood travel and so forth. The Geological Survey has now in process of preparation a final report on the hydrologic features of this flood, a report which it is anticipated will be well worthwhile in spite of the meager funds which have been obtained for the project. It is to be regretted that a larger part of the data on flood runoff to be presented in this report is not based on gaging station records and that consequently its value as a contribution to hydrology and as a practical work of reference will be somewhat less than it otherwise should be.

It is important for every hydraulic engineer to appreciate that while this particular record breaking flood occurred in central and southern New York, there is no physical reason why a similar flood or one even greater may not occur in any other part of the state. Whatever experience therefore which may be drawn from this flood will apply elsewhere as well. It is manifestly impossible to provide complete protection against floods of this order, but certain steps can always be taken to minimize the damage from similar future floods and the survey now under way under the direction of the Corps of Engineers will seek to suggest some such precautions.

The July flood clearly demonstrated, as did the 1927 flood in Vermont, the dangers of encroachment on natural stream channels of buildings and other structures. It demonstrated the importance of adequate bridge and culvert openings, which of course does not mean that such openings can be economically designed for such extreme flood flows as occurred in July. It demonstrated also the need, to which the New York State Planning Board has repeatedly called attention, for more stream gaging stations and more rain gages in order that full basic information on maximum and minimum stream flow and precipitation may be available. The better fortified we are with information and figures concerning floods and droughts, the better will we be prepared to meet such contingencies as they arise in the future.

(*Presented before the New York Section meeting, October 18, 1935.*)

MECHANICAL CONTROL IN WATERWORKS OPERATION

By A. H. R. Thomas

(*Superintendent, Water Works, New Toronto, Ontario, Canada*)

The controls in general use in the operation of a municipal waterworks range from simple to quite complicated controls. They may be considered as belonging to one or the other of the following groups: pressure, flow, or chemical feed controls.

The more common types of pressure controls are: pressure reducing valves, relief valves, altitude valves and pressure switches. Pressure reducing valves are used to reduce pressure in low lying districts.

Loss of head caused by valves and fittings in connection with pumps is inevitable, is responsible for waste of power, and affects economical operation of pumps. This waste of water, it must be remembered, is going on all the time the pumps are running, and it seems probable that a large number of engineers fail to realize its importance.

The old form of flap check valve or multiple disc check in large sizes shows appreciable losses.

Another bad feature is that due to design, excessive wear takes place rapidly on the hinge and pin which tends to drop the gate slightly and impair its tightness. The gate has to swing through practically a full 90° from fully open to closed, and on short pipe lines where a quick reversal of flow takes place when the pumps are shut down, results in severe slamming and pressure rise.

Tests have demonstrated that at a velocity of 7 feet per second, which is a normal pipe line velocity close to the pumps, the tilting disc check shows a loss of just over 0.2 feet, whereas in a standard check of the hinged gate type the loss would be around 1.5 to 1.7 feet.

Apart from the lower loss of head the valve disc has roughly half the distance to fall from fully open to closed, and is therefore quicker acting and overcomes slamming. It is "stable" in the open position and due to generous design of bearing surfaces and less movement has a long life.

Altitude valves are used to maintain a definite level in stand pipes

or reservoirs. When the level of the water reaches the desired elevation the valve closes, and when the level drops below a certain point, or the pressure in the main reaches a desired minimum, the valve opens.

Altitude, pressure reducing and relief valves are very similar except for the way in which they are connected. They can be controlled by pressure on a large diaphragm the movement of which is directly transmitted to the valve, or they may be controlled by an auxiliary or pilot valve. In small valves the first method is more usual, but in larger ones, say of 4 inches or over, the auxiliary or pilot operated type is more generally used, as it is not possible to operate a large valve directly and obtain any degree of accuracy unless the pressure be very low. In pilot operated valves, the pilot or auxiliary valve is controlled by a diaphragm or bellows, to which the controlling pressure is connected. This opens or closes the pilot which controls the large valve by admitting or exhausting water to or from a large diaphragm chamber or to a hydraulic cylinder depending upon the design of the valve. An ordinary gate valve can be controlled in this way. As the pilot valve has only a small movement, the balancing springs do not have to be compressed or expanded to any great extent, and therefore a very close control is obtained. By proper selection of the pilot almost any degree of sensitive control can be obtained. By connecting a number of pilot valves several functions can be obtained from one main valve. The pilots can also be arranged in such a way that the valves can be controlled manually when emergencies demand it. It is also possible to operate valves of this type by remote control manually or automatically.

Automatic pressure switches are used to stop and start pumps, and are controlled by the pressure in the main at the pumphouse or by the elevation of the water in the suction well, stand pipe or reservoir, the levels of which it is desired to control. When switches of this type are operated by the pressure in the main at the pumphouse they should be so designed that the surge, occasioned by starting the pump, does not immediately stop it. A simple form of switch is made from a pressure gauge. The gauge needle is removed from the spindle and in its place a sleeve is sweated on. From and at right angles to this sleeve projects a small pin which passes through a slot in a loose sleeve to which the gauge needle is attached. The pin is set to carry the gauge needle just past top center at either opening or closing of the contacts. The length of the slot in the loose sleeve

determines the pressure range. A better arrangement is to use a Mercoid switch to open and close a solenoid circuit. These switches can be arranged for emergency manual control which may or may not be from a remote point.

Flow controls are used in a filter plant for effluent rate control, and for wash water or backwash control. Effluent rate controllers may be operated directly or by means of a pilot valve. Direct operated controllers are usually simpler, and are therefore very often preferred in the smaller plants, but conditions largely determine the proper type to use.

With the improvement in motors and telemetering, electrically operated effluent controllers and filter gauges are available. In some filter plants a master control is installed which makes it possible to change the rate of flow through all the controllers at once. The master control can be made automatic by using the water in the clear well as the basis of control. Backwash control is usually accomplished by means of a rate of flow controller, or an ordinary pressure reducing control in the main wash water line. There appears to be no reason why the backwash should not be automatically controlled from the pressure in the underdrains or the difference between this pressure and that on the filter using the individual wash water valve for the purpose of control and controlling each valve from a pilot, the whole started and stopped from the operating table. The advantage of this method is that the quantity of wash water would be regulated by the actual pressure in the underdrains of the filter being washed. The rate would not be exceeded in any part of the bed if this pressure were accurately controlled. Should part of the bed break first, this part would take the whole of the wash water if a flow controller determine the quantity of water, but in the event of pressure control being used the velocity through any part of the bed could never exceed that corresponding to the control pressure.

Altitude valves may be used for water level control, but hydraulically operated gate valves, especially in the larger sizes, are more often used as they are less expensive. With the improvements that have been made in this type of control, the level can be maintained within very close limits.

Automatic shut off valves operate on excess flow, and are similar in principle to pilot operated rate of flow controllers. The differential pressure across some device which varies with the rate of flow or velocity is used to close the valve when such differential pressure has

reached the desired minimum or maximum. Devices of this kind are used to close the valve when the pump stops or when a break occurs in the main past the valve.

Automatic chemical feed control is used for the control of chlorine feed, for the feeding of solutions, or for feeding dry chemicals. The feed is in proportion to the rate of flow, whether these flows be large or small. The control can be taken from a small positive displacement meter, a rotary or turbine meter, an orifice, or from a venturi tube.

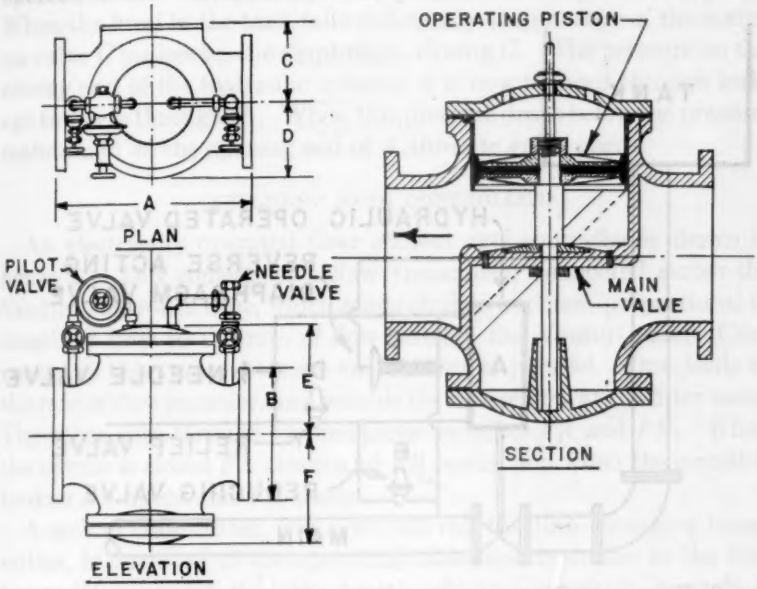


FIG. 1. PILOT OPERATED REDUCING VALVE USED TO MAINTAIN DESIRED PRESSURE IN MAIN WASH WATER LINE

PRESSURE REDUCING VALVE

A water pressure reducing valve operated by a pilot is shown in figure 1. The valve is operated by a hydraulic piston. Water is taken from the high pressure side through a needle valve (shown in the side elevation) into the hydraulic cylinder. From this cylinder it passes through a pilot reducing valve into the low pressure side. The pilot reducing valve is set so as to close when the desired pressure is reached on the low pressure side. The rise in pressure on the low

pressure side having closed the pilot valve, pressure builds up in the hydraulic cylinder and closes the valve. The needle valve is to regulate the closing rate of the main valve. When a drop in pressure on the low pressure side opens the pilot valve, water flows out of the hydraulic cylinder faster than it flows in, this causes the pressure in the cylinder to drop and the valve opens. A valve of this type can be installed in the main wash water line between the wash water tank and the first filter, and will maintain a fixed pressure in the main wash water line. It will not, however, unless reset each time, maintain the same pressure in the underdrain system of each filter. This is obvious

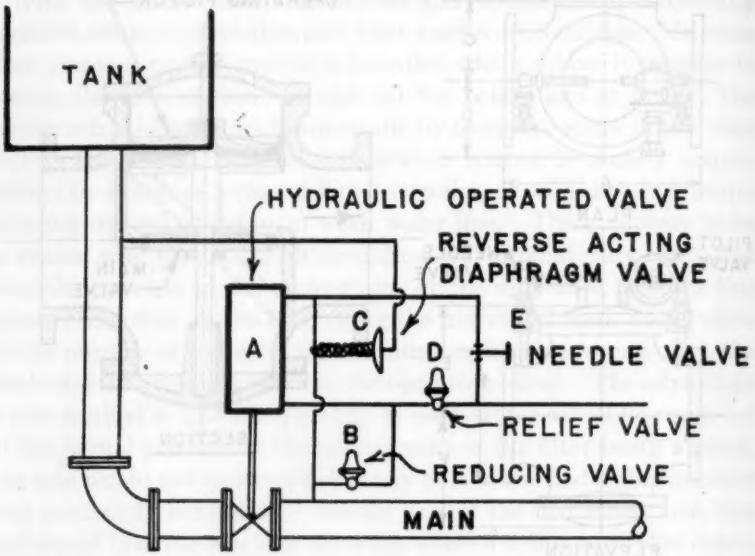


FIG. 2. HYDRAULIC OPERATED GATE VALVE ARRANGED AS ALTITUDE VALVE

on account of the varying distance and sometimes varying number of fittings between the individual filter and the control valve. Such a valve, however, is a great help in the control of wash water.

HYDRAULICALLY OPERATED GATE VALVE

Figure 2 shows a hydraulically operated gate valve, arranged to operate as an altitude valve. *A* is a hydraulic cylinder. *B* is a reducing valve set to maintain the pressure on the opening side of the hydraulic cylinder at half the normal main pressure. *D* is a relief valve, relieving to waste and set slightly above reducing valve *B*.

C is a reverse acting diaphragm operated valve, the pressure on the diaphragm, which governs the movement of the valve, being taken from the tank riser pipe. Valve *C* is set so that when the tank is full it opens and allows full main pressure upon the closing end of the hydraulic cylinder *A*. This pressure being double the pressure on the opening end of the cylinder the valve closes, water from the opening end exhausts through relief valve *D*. *E* is a needle valve controlling the rate of flow from the closing end of the hydraulic cylinder to waste. It is opened a very slight amount, so that when valve *C* opens, pressure can build up in *A* sufficient to close the gate valve. When the level in the tank falls sufficiently the pressure of the spring on valve *C* pushes up the diaphragm, closing *C*. The pressure on the closing end of the hydraulic cylinder *A* is now reduced through leakage to waste through *E*. When this pressure drops below the pressure maintained on the opening end of *A* the gate valve opens.

EFFLUENT RATE CONTROLLERS

An electrically operated filter effluent rate controller is shown in figure 3. This consists of a flow transmitter connected across the Venturi controller tube, which sends electric impulses proportional in length of time to the rate of flow through the Venturi tube. Connected to this transmitter are two circuits in parallel. One leads to the rate of flow recorder, and records the rate of flow at the filter table. The other goes through two magnetic switches *FA* and *FB*. When the circuit is closed *FA* closes and *FB* opens, and when the circuit is broken *FA* opens and *FB* closes.

A second transmitter, which we will call the Rate changing transmitter, is mounted at the operating table, and is similar to the flow transmitter, except that the length of impulses which it sends is governed manually by the setting knob. The impulses pass through two magnetic switches *RA* and *RB*, so that when the circuit is closed *RA* opens and *RB* closes, and when the circuit is broken *RA* closes and *RB* opens. The rate of flow corresponding to the length of the time impulse for which the transmitter is set is indicated on the rate changing transmitter for the information of the operator. The operation is as follows:

The operator sets the rate changing transmitter for a desired rate of flow, which results in *RA* being closed for a portion of the time cycle corresponding to the desired rate of flow. We will assume that the desired rate is higher than the rate at which the filter has been

operating. The circuit on the rate changing transmitter is, we will assume, closed for 50 percent of the time cycle. We will further assume that the flow transmitter has been operating at a rate such that its circuit is closed for only 40 percent of the time cycle.

In adjusting the controller, the two transmitters are of course synchronized, so that their time cycles start at the same moment.

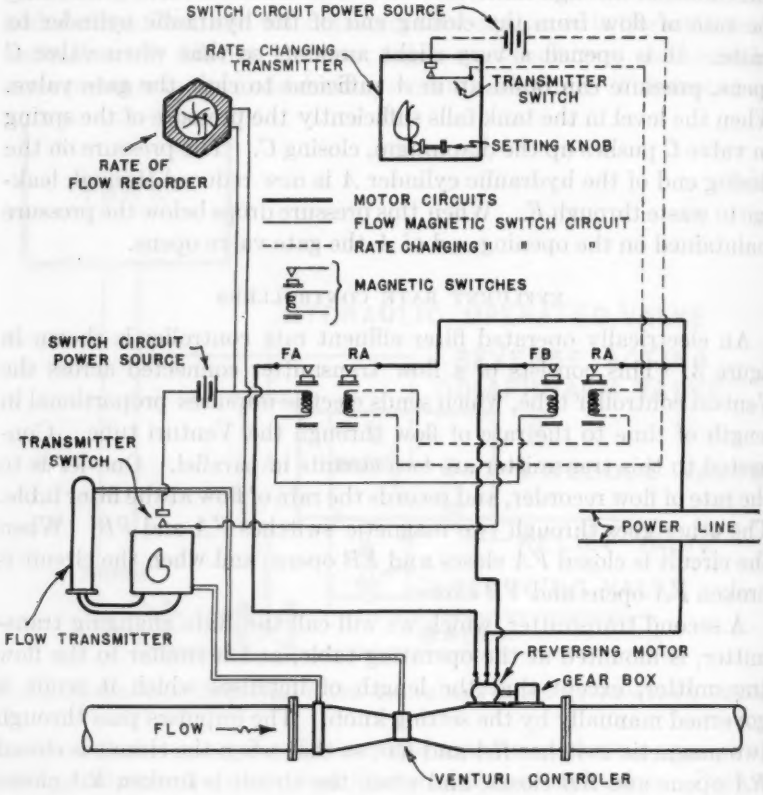


FIG. 3. ELECTRICALLY-OPERATED FILTER EFFLUENT RATE CONTROLLER

Under the conditions assumed, magnetic switch *FA* is closed for 40 percent of the time cycle, but switch *RA* in the same circuit is open for 50 percent of the time cycle. Therefore, that circuit remains open continuously. When the circuit of the flow transmitter is broken *FB* closes, and it is closed for 60 percent of the time cycle. When the circuit in the rate changing transmitter is broken *RB* opens,

but only for 50 percent of the time cycle, so that power flows through *FB* and *RB* for 10 percent of the time cycle, and this power operates the motor which through a gear box opens the controller valve increasing the flow. As the flow increases the length of time the circuit from the flow transmitter is made increases until it is 50 percent on and 50 percent off. When this condition is reached the power circuits remain open continuously and the controller valve remains stationary. Any change in the rate of flow through the controller brings the two impulses from the two transmitters out of step, which results in the motor running in one direction or the other to open or close the controller valve until the desired rate of flow is restored.

AUTOMATIC CONTROL OF WASH WATER

Figure 4 shows a method of automatically controlling wash water from the difference in pressure between the underdrains and the water above the filter. A diaphragm pendulum unit, controlling a pilot valve, is connected to the filter above the sand and to the underdrain system. The supply to the pilot valve is connected to the wash water four-way operating valve on the filter table in place of the line which usually runs to the opening end of the hydraulic cylinder. In the line from the four-way operating valve to the closing end of the cylinder is placed a check valve which allows water from the four-way valve to enter the cylinder but closes when this flow is reversed. This line is of sufficient size to ensure quick closing. The outlet to waste from the opening end of the hydraulic cylinder passes through a diaphragm control valve to waste. This line is also large enough to allow the valve to operate quickly. The lines from the pilot valve are small, as the pilot valve itself is small, and they are not subject to a large flow. The operation is as follows:

After the effluent valve is closed and the bed is ready for washing, the operator turns the handle of the wash water four-way valve to open. This admits water pressure from the supply header to the pilot valve, and also to the diaphragm control valve, which closes, preventing leakage to waste through the large line. The pilot valve admits water slowly to the opening side of the hydraulic cylinder controlling the wash water valve. Water from the other end of the cylinder exhausts through the pilot valve to waste. As the valve opens slowly pressure in the underdrain system builds up slowly. When this pressure rises sufficiently the pilot valve goes to neutral,

and the main valve movement ceases. The pilot valve regulates the opening and closing of the wash water valve so that the desired difference of pressure is maintained. When washing is completed

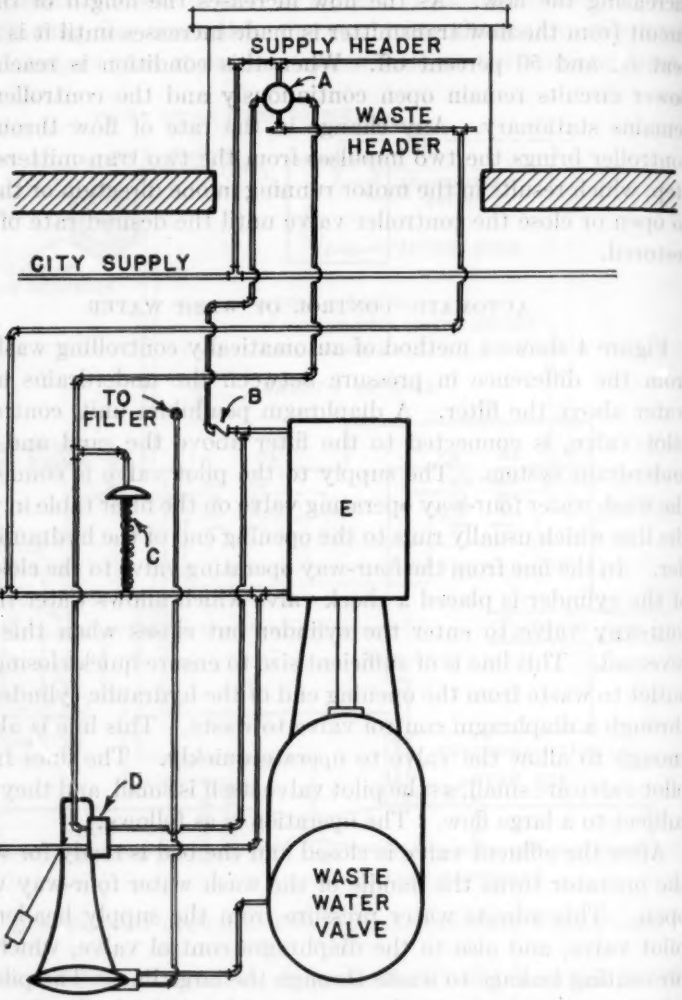


FIG. 4. AUTOMATIC CONTROL OF BACKWASH BY DIFFERENCE IN PRESSURE IN UNDERDRAINS AND HEAD ON FILTER

A = 4-way valve controlling wash water valve; *B* = check valve; *C* = diaphragm control valve; *D* = pilot valve operated by differential pressure across diaphragm; *E* = hydraulic operated gate valve.

the operator moves the handle of the four-way valve to closed. This removes pressure from the supply line to the pilot valve, and from the diaphragm control valve, which therefore opens allowing the exhaust from the hydraulic cylinder to pass quickly to waste. Water is admitted through the large line from the four way valve to the closing end of the cylinder, and due to the opening of the check valve the flow is rapid and the wash water valve closes quickly, minimizing the waste of wash water after washing is completed. As soon as the valve is closed the operator returns the four-way valve to neutral.

Figure 5 shows an arrangement similar to that shown in figure 2 except the valve C is operated from differential pressure across the

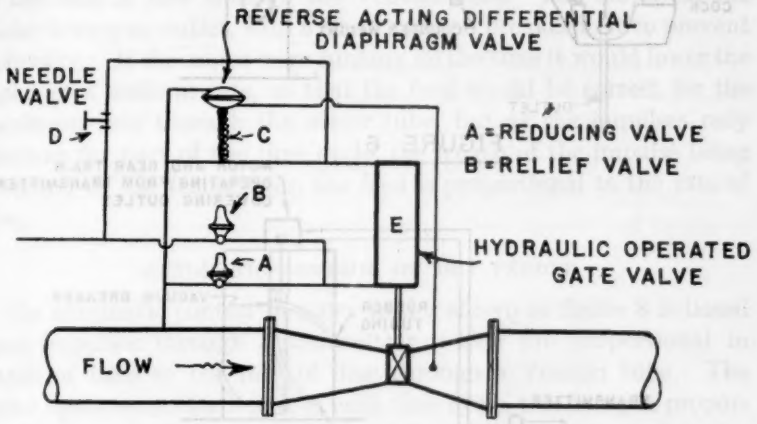


FIG. 5. GATE VALVE ARRANGED TO CLOSE ON EXCESSIVE VELOCITY

valve, which is placed between a decreaser and an increaser. This has the effect of a Venturi tube, so that the pressure taken from a smaller diameter on the down stream side of the valve is less than that taken from the full diameter of the pipe on the upstream side of the valve. The reducing valve A and relief valve B maintain half the main pressure on the opening end of the hydraulic cylinder E. When the rate of flow through the valve becomes greater than the desired amount due say to a break in the main beyond the outlet side of the valve, the differential pressure across the diaphragm of valve C which is a reverse acting valve, increases, opening the valve. This admits full main pressure to the closing end of the hydraulic cylinder shutting off the main valve.

AUTOMATIC CONTROL OF SOLUTION FEEDER

Figure 6 shows a method of automatically controlling a solution feeder using a Venturi meter to regulate the flow of solution. A cord around a pulley on the main shaft of the Venturi meter register, leads

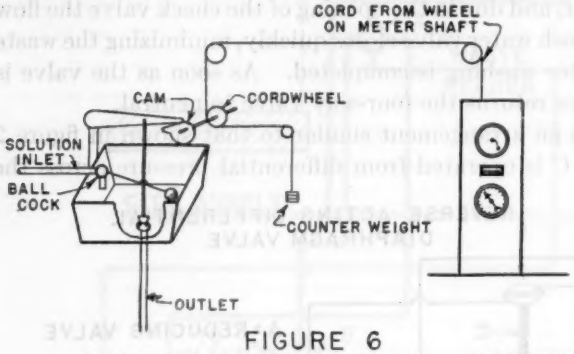


FIGURE 6

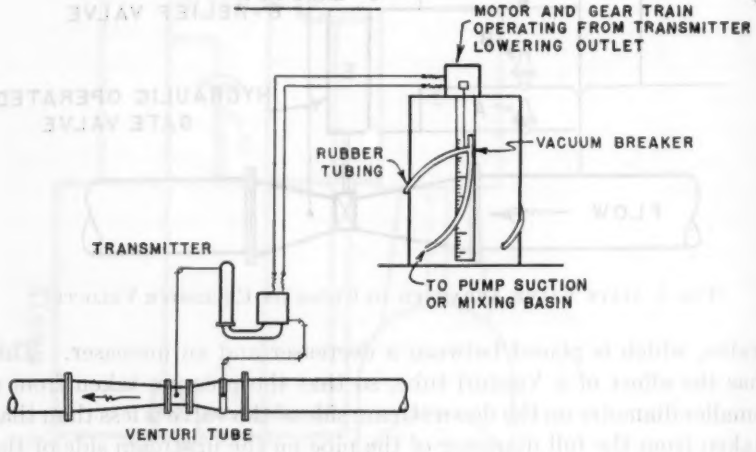


FIGURE 7

FIG. 6. AUTOMATIC CONTROL OF CHEMICAL SOLUTION FROM VENTURI METER
FIG. 7. AUTOMATIC CONTROL OF GRAVITY FLOW SOLUTION FEEDER

to a pulley on a shaft at the orifice box. On this shaft is mounted a cam, with cam follower on a lever. The valve stem which controls the opening of the orifice in the bottom of the orifice box is connected to this lever. Any change in rate of flow through the Venturi tube causes a change in the position of the valve controlling the flow

of solution from the orifice box. The head on the orifice is kept constant by means of a float operated ball cock. An overflow (not shown) is usually provided in case the ball cock sticks.

Automatic control applied to a gravity flow solution feeder is shown in figure 7. This type of solution feeder is suitable for feeding by gravity into a mixing chamber or a pump suction. A Venturi tube through which the water to be treated flows is connected to a transmitter which sends an electric impulse to a motor on the feeder. These impulses are sent out at regular intervals, so many intervals per hour or minute depending upon the design of the transmitter. The duration of the impulse sent by the transmitter is proportional to the rate of flow through the Venturi tube. The motor on the feeder lowers an outlet, with a small vacuum breaker in it to prevent siphoning. If the motor were running all the time it would lower the outlet at a uniform rate, so that the feed would be correct for the maximum rate through the meter tube, but as the impulses only continue for part of the time cycle, the length of the impulse being governed by the transmitter, the feed is proportional to the rate of flow.

AUTOMATIC CONTROL OF DRY FEEDER

The automatic control of a dry feeder shown in figure 8 is based upon impulses through a transmitter, which are proportional in length of time to the rate of flow through a Venturi tube. The feeder operates during a part of each time cycle, this being in proportion to the rate of flow through the tube. The solution tank under the feeder has a retention period of about ten minutes which takes care of that part of the time cycle during which the feeder is stationary.

It is possible to use the transmitter to vary the speed of the motor in direct proportion to the rate of flow, so that the operation of the feeder is continuous rather than off and on. This of course would increase the cost of the equipment, and it does not appear to be necessary, as the arrangement shown is apparently giving satisfactory results.

A chemical feeder mounted directly on top of a meter between the intermediate gear and the register is shown in figure 9. The meter operates a pilot valve so many times per minute this being proportional to the rate of flow. The pilot valve regulates the admission of water from the main, first to one side and then to the other of a

diaphragm motor. The exhaust from the motor runs to waste. The diaphragm motor drives a small diaphragm pump, the case and valves of which are rubber. This pump will feed hypochlorite or chlorine

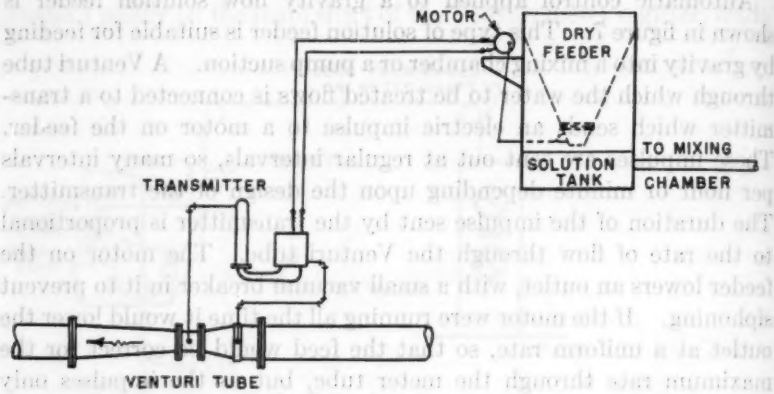


FIG. 8. AUTOMATIC CONTROL OF DRY FEEDER

Transmitter operates feeder motor for percent of a time cycle proportional to rate of flow through meter tube. Time cycle is selected to suit conditions and is short.

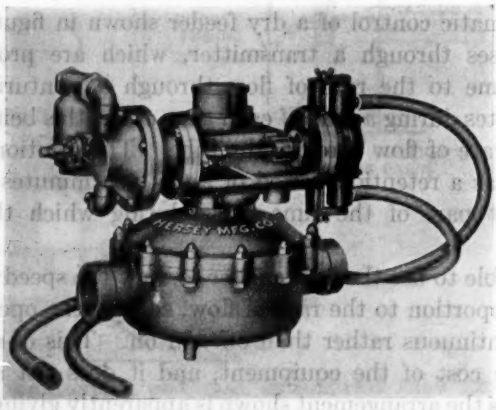


FIG. 9. CHEMICAL FEEDER MOUNTED ON TOP OF A METER

solution into the main in direct proportion to the rate of flow. The pump takes the solution from a stoneware tank, the strength of the solution being adjusted to suit conditions. As the length of pump stroke can be changed, the delivery of the pump can be varied for the

same number of strokes per minute. The quantity of solution fed per stroke can be increased or decreased by this means, and the meter regulates the number of strokes according to the rate of flow.

Figure 10 shows a dry feeder controlled automatically by a meter of the Turbine or any other displacement type. A gear box placed between the intermediate gear train and the register of the meter is operated therefore in proportion to the rate of flow through the meter. A gear train in the gear box operates a pilot valve a number of times (usually from 10 to 40 strokes) per minute. A hydraulic cylinder mounted on the dry feeder is operated by water taken from the main under pressure and is controlled by the pilot valve. The hydraulic

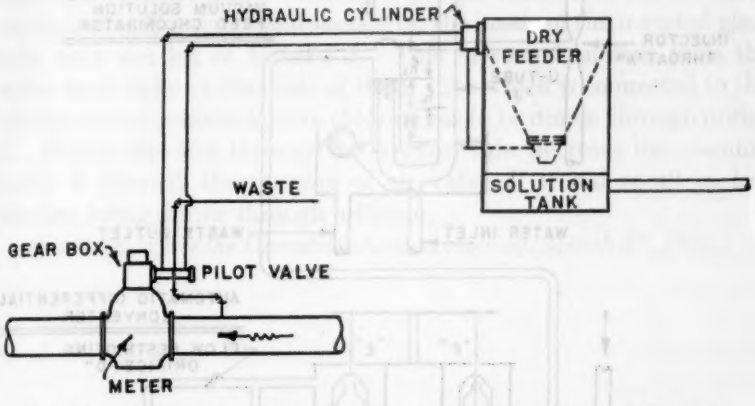


FIG. 10. AUTOMATIC CONTROL OF DRY FEEDER FROM TURBINE METER
Meter operates pilot valve, which strokes hydraulic cylinder and feeder, in proportion to flow.

cylinder operates the feeder. When the flow of water through the meter increases, the number of strokes of the pilot valve and consequently of the hydraulic cylinder increases, and with a decrease in flow a corresponding decrease in the number of strokes takes place, thus giving feed control in proportion to the rate of flow.

AUTOMATIC CHLORINATOR

The automatic chlorinator shown in figure 11 consists of a converter the purpose of which is to convert the differential pressure from a Venturi tube into a differential vacuum. Connected to and controlled by this converter is the chlorinator which feeds chlorine solution to the water which is to be chlorinated.

The Venturi inlet is connected to the top of chamber E and the throat connection leads to the bottom. A by-pass connects the inlet or high pressure pipe to the throat or low pressure pipe and in this by-pass at C is inserted an orifice thereby reducing the differential, another orifice at D further restricts the flow. The inverted cup in

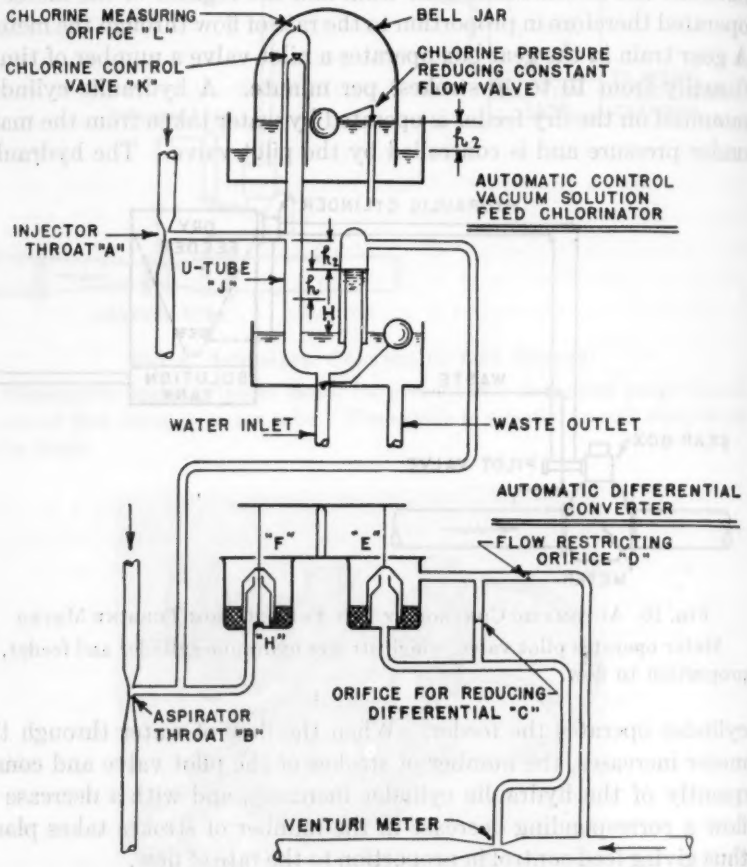


FIG. 11. OPERATION DIAGRAM OF AN AUTOMATIC CHLORINATOR

chamber E is suspended from a beam supported on a knife edge to the other end of which is connected the inverted cup in chamber F, the top of chamber F is open to atmospheric pressure and the bottom is connected to aspirator B and also to atmospheric pressure through air inlet valve H. From the connecting pipe between chamber F and

aspirator *B* a connection is taken to the inside of an inverted glass tube the bottom end of which is open and submerged in water, the top being closed. One leg of U tube *J* rises inside this inverted glass tube, and the other leg is carried up inside chlorine meter tube *L* and from this leg a connection is taken off to injector throat *A*. The operation of this equipment is as follows:

An increase in flow through the Venturi tube increases the differential across inverted cup *E* causing this cup to move downwards and this downward movement is transmitted through the beam to cup *F* which moves upwards, and this upward movement closes or partially closes air valve *H*, this results in aspirator *B*, which is drawing air from the inside of cup *F* creating a partial vacuum inside *F*. This vacuum causes an increased suction on the inside of the inverted glass tube over one leg of U tube *J*. This increased suction pulls the water level down in that side of the U tube which is connected to the injector thereby causing more chlorine gas to be drawn through orifice *L*. Should the flow through the Venturi tube decrease less vacuum inside *F* through the opening of air valve *H* would result in less chlorine being drawn through orifice *L*.

(*Presented before the Canadian Section meeting, March 27, 1935.*)

PIONEERING IN THE WATER WORKS FIELD

By Morgan D. Hayes

(*City Engineer, Rochester, N. Y.*)

In selecting a subject for this paper, the writer had in mind the various exhaustive reports that had already been made on the subject of the Rochester Water Works, ranging over a long period of years. The data for these papers were assembled and arranged for presentation either as reports to the Rochester City Council or as papers read before local engineering societies and water works conventions. Among the list was a paper read in 1932 before the Rochester Academy of Science by Edwin A. Fisher, then City Engineer of Rochester, covering the history of the Rochester Water Works in its first century, commencing in 1834.

Another paper covering the same subject, and prepared by Irving Matthews, formerly Superintendent of the Rochester Water Works, was read before the New York section of the American Water Works Association at Rochester in 1927. Another elaborate report was published in 1913 by John F. Skinner, Principal Assistant Engineer, covering conduits Nos. 1 and 2 of the Hemlock Lake System.

With these data available, I take no credit for anything original in the following paper, but as a water works engineer I have been greatly impressed by the number of interesting facts brought out in the early history of these works. With this in mind, it appeared worth while to stress again those particular instances wherein Rochester appears in the rôle of *"trail blazer"* or pioneer in the water works field.

In order not to repeat the many details already covered by the former papers, I have confined myself to the more salient features and with particular emphasis on those of most historic importance. Many long strides forward have been taken since the events here narrated.

GRAVITY CONDUIT NO. 1

The first major pioneering work in the Rochester Water System was the installation of its gravity conduit No. 1 from Hemlock Lake, and of the three gravity pipe conduits now supplying the city, this line is historically the most interesting.

The two novel features connected with the line were: first, the type of pipe used, and second, the formula employed to estimate its probable capacity.

The elaborate series of tests afterwards made on the line for flow coefficients, together with the extended investigations on pipe covering and corrosion, have now become classic in water works literature.

When this line was installed in 1873, the wrought iron pipe used was the first pipe of its kind east of the Rocky Mountains, the only other line of similar character being several miles of wrought iron plate pipe laid by the Spring Valley Water Company, of San Francisco, Cal. Eastern engineers looked with suspicion on this type of construction.

The line is $28\frac{1}{4}$ miles long and is compound, having the 36 inch diameter pipe at the south end laid on a gradient which drops only 23 feet in 10 miles; the other portion is 24 inches in diameter with a gradient which drops 232 feet in 18 miles. All the 36 inch and a portion of the 24 inch is constructed of wrought iron plate only $\frac{3}{16}$ inch thick, shop riveted and shipped in 28 foot lengths and field riveted to make 84 foot laying lengths. The ends are riveted to cast iron hub and spigot pieces and the joints are made with lead.

In estimating the carrying capacity of this conduit, Mr. J. Nelson Tubbs, designer and then resident engineer, was confronted with a problem containing many engineering features which at that time were not well understood.

He estimated the carrying capacity of the whole line at 7 m.g.d., using the old Weisbach formula for uniform diameter, modified to cover his compound line.

If he had had a Williams and Hazen table of flow coefficients at hand he could have saved many days and perhaps some sleepless nights of worry over hydraulic formulae, and reached a much more accurate solution of his problem.

After the line was installed, various flow tests were made by L. L. Nichols, his assistant engineer, in 1876, partly by observation of displacement in Rush Reservoir, and partly by calculations, using a coefficient recommended by Eytelwein, Downing and D'Aubuisson, with results ranging from 7 to 9 m.g.d.

In 1890 Mr. Emil Kuichling, then Chief Engineer of Water Works, checked over these calculations, and after making certain corrections in the coefficient used, stated that the capacity of the line was approximately 9 m.g.d.

In 1889 Mr. Clemens Herschel, engineer for the East Jersey Water Company, designed a 48 inch riveted steel conduit for Newark, N. J.,

employing Mr. Kuichling, as his assistant and using the Rochester data on conduit No. 1 in his calculations.

Although the Newark conduit failed to carry the estimated capacity of 50 m.g.d. and although Mr. Herschel blamed his failure on inaccurate data furnished by the Rochester flow tests, the fact remains as stated by Mr. Herschel, that the gaugings made on conduit No. 1 in 1877 furnished the "basis for all subsequent calculations on the flow of water through long lines of large riveted pipe" up to that time, about 1890.

Conduit No. 1 was a real pioneer, and has furnished much waterworks history. Through the years 1895 to 1897 the Engineering News carried articles by many of the most eminent hydraulic engineers of this country on the subject of carrying capacity of large conduits, with speculations as to the proper coefficients to use in the well known Kutter formula, viewed from the results of the flow tests on old conduit No. 1.

The Kutter formula was first published in 1869, only four years before the design of the Rochester conduit.

Following is a quotation from the report of the Executive Board of Rochester, in 1879, made by Mr. J. Nelson Tubbs, Chief Engineer, covering this line:

"I think our conduit line is the longest of its kind in this country and perhaps in any other. Great interest has been manifested ever since our works were completed, by those interested in hydraulic science, that careful experiments should be made of the flow of water in our conduit with a view to the correction of hydraulic formulas relating to that subject. The results of such experiments carefully conducted would be received by the scientific world as most valuable contributions to science, and it may perhaps be regarded somewhat in the light of a duty, that those having control of unusual facilities for making such examinations should do so, for the general good."

Note: Old wrought iron section of 36 inch line abandoned in 1934. 60 years old.

HOLLY FIRE SYSTEM

Rochester has three water systems.

1. A potable domestic supply by gravity from two upland lakes, supplying the major part of the City. Municipally owned.

2. A potable domestic supply pumped from Lake Ontario and serving five of the City Wards. Privately owned.

3. An independent direct high pressure fire system known as the

"Holly System," pumped from two mill races along the Genesee River, with the City. Municipally owned.

As the domestic system had a gravity supply with pressures not generally sufficient for fire purposes, a direct pressure fire system was installed the same year 1873, making this a real pioneer in the field of direct pumping systems.

It served the densely built sections of the City, and when first used, it was cross connected in several places with the domestic system so as to serve the whole City, although its source was from Brown's Race.

The distribution lines ranged from 4 to 24 inches in diameter, and carried a hydrant pressure during fires of 135 pounds per square inch, under direct pumping.

The pumping equipment was both novel and original, and was built and installed by Mr. Birdsill Holly, of Lockport, New York, whose name was, ever afterwards, applied to the system. The original equipment consisted of two quadruplex pumping engines of 2 m.g.d. capacity each, arranged for operating either by two water turbines under 90 foot head, or by an independent steam engine.

There was also one steam driven quadruplex pumping engine of 3 m.g.d., also two Holly rotary pumps of 2 m.g.d. driven by a steam engine.

The plant was located on Brown's Race, in the City.

Immediately after installation a fire test was made in 1874, using a series of hydrants located on Main Street, from the Erie Canal on the West to North Street, on the East.

An account of these fire tests, written by J. Nelson Tubbs, then Engineer of the Rochester Water Works, states as follows:

"It is believed that this was the most remarkable exhibition of large streams ever made in any country, and as such, it attracted widespread attention from hydraulic engineers, compelling the introduction of larger factors in the hydraulic formulas used to determine the results to be obtained from large streams, with liberal size pumping mains."

TEST NUMBER	NUMBER OF STREAMS	SIZE OF STREAM	HEIGHT OF STREAM	PUMP PRESSURE PER SQUARE INCH
		inches	*feet*	*pounds*
1	14		131 to 152	120
2	30		131 to 152	135
3	1	2	210 vertical	165
4	1	3	286 vertical	175
5	1	4	294 vertical 465 horizontal	175
6	1	5	256 vertical	140

During the first test a change-over was made from water power to steam power without being noticed by the observers.

This was a remarkable demonstration of mechanical efficiency considering the time in which it was made, 61 years ago, and it is illuminating to compare the date of this installation with that of other independent fire systems in some of our largest cities:

Rochester	1873
Buffalo	1897
Philadelphia	1902
New York City	1905–1908
Pittsburgh	None

The Holly pumps were replaced by electrical centrifugal units in 1925, after 50 years of service.

TELEGRAPH SYSTEM TO HEMLOCK LAKE

At the time of the construction of the Hemlock Lake water system, with its 28 mile gravity conduit to the City, this territory was in the "mud-road and horse and buggy stage," and it took a full day to negotiate the distance between Rochester and Hemlock Lake by the usual method of transportation, there being no railroads covering this entire stretch of country.

As the first commercial telegraph line in the world, known as the New York, Albany and Buffalo Telegraph Company, had been projected and constructed by Henry O'Reilly of Rochester, opening its Rochester office in 1845 under the Morse patents, it was only natural, therefore, that the officials of the local water company should take advantage of this quick method of communication in controlling their water system. They built, therefore, a private telegraph line from the City to Hemlock Lake in 1877, using the Morse recording instruments.

This was, therefore, the pioneer telegraph[1] system in the operation of water works.

In 1879, the Bell Telephone Company opened a Rochester office and

[1] The word "telegram" was coined in Rochester.

In 1853 a Mr. E. H. Smith of Rochester wrote the "Albany Journal" that he wished to introduce a new word into the English language, namely, "Telegram." He then described the etymology of the word, showing its formation from two Greek roots, which translated mean "Writing at a Distance."

From that period on the word "Telegram" became of universal use in the commercial system.

immediately the Bell type of receiver was substituted for the Morse Recording instruments. The Hemlock line then became what was said to be the longest *telephone* line in actual use at that date, and was the source of great curiosity and discussion on the part of the general public; this was pioneering in a *double sense* on the part of the water company.

METEOROLOGICAL AND HYDROLOGICAL DATA

Not many cities in this country have as old or as complete a series of meteorological records, as is possessed by the Rochester Water Bureau.

Beginning with the construction of the Hemlock Works in 1873 almost continuous records have been kept, the period of years in each case being as follows:

Record	*Years*
Rainfall	55
Temperature	55
Evaporation from water surfaces	39
Run-off	39

In a 48 year period at Hemlock Lake, the average yearly rainfall was about 28½ inches.

The average figures for evaporation on Highland Reservoir for a period of 31 years is $31\frac{8}{10}$ inches or only ¾ inch less than the rainfall recorded at Rochester.

Rochester's evaporation records are especially interesting, and were instituted by Mr. Emil Kuichling in 1896, and have been continued to date, almost 40 years.

The method used was the placing of two indurated fiber tubs about 15 inches in diameter and 6 inches deep, one floating in the reservoir and the other placed on shore, and each filled every day with a known quantity of water. At certain intervals the tubs are removed and the amount of water remaining determined, the difference in each case, being the loss by evaporation.

In winter, a special type of tub is used in the water, and as this is apt to freeze, it is therefore weighed and replaced twice a day.

Temperatures of air and water, as well as rainfall and velocity and direction of wind, are carefully observed at the same time, as all these factors affect the result.

As reliable evaporation figures are not very common, the Rochester records have been extensively used in meteorological tables.

The run-off records are actual measurements over weirs or through meters, and have been very valuable in hydraulic calculations, not only in our own, but in other water districts of New York State. Most of them have been taken in collaboration with, and following, the standards of the U. S. Geological Survey Department of Washington, D. C.

APPLICATION OF IODINE TO THE ROCHESTER WATER SUPPLY

In 1923, considerable discussion arose throughout this country over the fact that goiter had become prevalent in many regions, and steps were being taken to prevent the spread of the disease in localities where iodine is lacking in the food and water.

In a survey made by J. F. McClendon, of the University of Minnesota, Rochester was found to be in a goitrous belt. After long discussion and many experiments, it was decided, as a remedy for the situation, to apply iodine to the water supply under the direction of the Commissioner of Public Safety, with the help and collaboration of Mr. Beekman C. Little, then Superintendent of the Water Bureau.

The method used was to apply a dose of about 16 pounds of iodine per day for the first week, and at longer periods for the balance of the time, which covered two to four weeks in the spring and fall of each year.

This practice continued from 1923 through 1932 and resulted in a very great reduction in the number of cases of goiter among the children and adults of the City.

I have been advised by Dr. A. M. Johnson, the present Health Officer, that the practice was only discontinued because of the increased general use of iodized salt throughout the City, as well as for reasons of economy.

I do not know to what extent Rochester's example was followed by other cities. I am only stressing the fact that Rochester tried the experiment first in its water supply.

ELECTROLYSIS

About the year 1890, the subject of electrolysis as a possible factor of damage to water systems was being agitated.

In that year Mr. Kuichling, the City Engineer, made the most extensive study of this subject that had been made in any City.

His report covered a summary of proposed remedies and investigations, which had been made in 25 other cities and was carried on in

conjunction with the local streel railway, the gas and electric and the telegraph and telephone companies over a period of several years.

It was found that no definite damage had resulted therefrom, except in the case of a few water services. Mr. Albert B. Herrick, a specialist on the subject, collaborated in these studies.

COBBS HILL RESERVOIR

Cobbs Hill Reservoir, within the city, was built under the design and direction of the city's engineering staff with Mr. Frederick P. Stearns, of Boston, as consultant, and the concrete drainage tunnel or gallery under the entire embankment, although not the first of its kind, is rather unusual in reservoir construction and well worth a visit.

GENERAL

Many interesting events attended the early "growing pain" era of the city's water systems.

Prior to 1873 when the present works were started, there were several attempts made to supply water systems; these varied from ponds or small dams with wood stave supply pipes, to artesian wells, all under private management and all proving inadequate for various reasons—physical, mechanical, or financial, so that an adequate and satisfactory system was not available until the Hemlock Lake supply came into being.

All in all, the history of the local plant now covers over a hundred years.

(Presented before the New York Section meeting, October 17, 1935.)

HOW ONE COMPANY MEETS ITS PROBLEMS OF METER READING, BILLING AND COLLECTING

By John C. Houston, Jr.

(Office Manager, Western New York Water Company, Buffalo, N. Y.)

The Western New York Water Company, supplying the suburban area adjacent to the city of Buffalo, is privately owned. Its system surrounds the City on three sides, and comprises approximately 290 miles of mains, serving a population of about 120,000, in the villages of Sloan, Depew, Lancaster, Williamsville, Blasdell, and Kenmore; the City of Lackawanna; and the towns of Hamburg, Orchard Park, West Seneca, Cheektowaga, Alden, Lancaster, Amherst, Clarence and Tonawanda. Water is taken from Lake Erie at a point just south of the city of Lackawanna in the unincorporated hamlet of Woodlawn, where it is filtered and supplied to the southern half of the territory from the main pumping station. In order to serve the balance of the territory efficiently, two main booster stations are located at strategic points, and there are three smaller auxiliary boosters to take care of peak loads.

The system is 100 percent metered, and of the 12,500 active accounts, all but 100 are either domestic or small commercial consumers. This large group of small consumers forms the basis of this discussion, because their problems are general, whereas those in the other group are individual and special, most of them being large industrial corporations, or municipalities who buy water wholesale and distribute it themselves. These accounts are billed every month, but the bulk of our consumers are on a quarterly billing basis.

Considered from the standpoint of both company and consumer, we believe that bills rendered once every three months are most satisfactory. A shorter billing period would materially increase the company's operating costs without bringing proportional benefits to the customer; and a longer period not only ties up revenue, but often means that a consumer is presented with a bill for much more than he cares to pay at one time. Our average quarterly bill is about equal in amount to the average monthly bill for other utility services.

To facilitate all routine work in connection with these accounts, we have divided our territory into three districts known as one, two and three. Each is a compact, geographical unit, and all are fairly equal in area, number of consumers and total revenue produced. District 1 is read and billed in January, District 2 in February, District 3 in March, and then the cycle begins again. We thus have an even distribution of work, inside and out, every month of the year, and are also able to maintain a fairly steady flow of cash, a decided asset to any business.

In connection with keeping this cash flowing, I am often reminded of the reply which I received from a veteran of the water works industry when it was all new to me and I was inquiring about maintaining good public relations. "My boy," he said, "It requires diplomacy and diplomacy has best been defined as the art of giving the other fellow your own way. In other words, remember that the customer is always right, but convince him that you are right, too."

Continuing the division of our accounts into units easy to handle, each district has been broken down into 25 routes, so-called because a meter reader can start at the beginning of any book and find the sheets all arranged in the order in which they are to be read. This order is the one that will save him the most steps, because it has been carefully worked out from maps and from actual experience in the field. Each route consists of approximately 160 accounts, and the sheets are in loose-leaf form, but bound by durable bakelite and metal covers. In using this heavy binder, our chief aim was the protection and preservation of the reading sheets, but judging by unofficial reports they seem to have a quieting effect on those dogs who think that meter readers make good meals.

The sheets in each book are numbered to allow the addition of new accounts in their proper place, and all of our office records (with the exception of our meter history cards) are filed in route number order, and we thus eliminate all necessity for cross-indexing and have instead a very simple but efficient filing system. The key to this system is a geographical index composed of 3- by 5-inch white cards grouped alphabetically by streets and numerically by house numbers. At the top of these cards is the address, and immediately below are shown the route number and the number of the meter supplying the property. On certain streets, all houses do not have house numbers, in which case those which have are arranged in numerical order, and on those which have not we write the customer's name and file them alphabetically behind the cards having house numbers. This file,

incidentally, is a great time saver, and any office, no matter what the size, would find it worth while installing. I have seen it opposed by old time book-keepers and cashiers who gave various excuses for not wanting it installed, but after it had been made up and placed in service they could not understand how they ever got along without it.

The meter reading books, except for a few days each quarter, are kept in the office, for they constitute the most important record that the company has of any consumer. Although no cash transactions are kept in them, all other pertinent items, such as turn ons, turn offs, meter changes, inspections of all kinds, and various miscellaneous bits of information are indicated on the individual sheets. After each route has been read, it is carefully checked at the office, and each consumption compared with previous usage at the property. If it is exceptionally high or low, an order is issued to re-read the meter. If the reading is found to be correct but high, a thorough check of the plumbing is made by the inspector who makes a verbal report to the occupants of the building, and a written report to the office. This second, check reading not only aids the customer in preventing further wastage of water if our inspector finds there are leaks in the plumbing, but it also aids materially in handling consumers' complaints because you have anticipated them before they come in, and people are usually satisfied if you can give them a reasonable explanation for a high bill. If there is none, and the customer is dissatisfied, a test of the meter is made in service. This is done without charge, and for all practical purposes is accurate enough, but if a more complete test is desired, we will remove the meter to our shop where, in the complainants, presence, we will test it at various rates of flow. If it is found to be in error, an adjustment is made in accordance with the Public Service Commission regulation governing such cases, but if it is found to be correct, a charge of $2.00 is made for the service. This does not cover the cost incurred by the company, but it prevents the needless removal of meters and makes the customer look around and usually find the real reason for his high bills. To aid in this, the company is always willing to send out a qualified inspector.

In those cases where the meter reader finds on a re-call that the consumption is low, he turns on a small stream of water and then checks to see if the meter is registering. If it is not, he so indicates on the sheet. Otherwise he questions the occupants as to vacancies during the past quarter and tries to determine the cause for the drop in consumption. If there seems to be no reason for it, a report is

made to the office and the meter is ordered out of service and taken to the shop where it is tested and any worn parts replaced by new ones. This is in addition to the periodic checking which we do in compliance with the Public Service Commission rules and regulations governing the testing of water meters. These rules, however, did not differ greatly from our established practice before they became effective, because we know that the meters are our cash registers, and that we cannot be too careful in our maintenance of them. Some of our customers, I am afraid, think that we maintain them entirely too well, but I will say that I have never taken anyone out to our shop to witness a test of their meter that they did not come away satisfied that they were receiving a fair deal, and impressed by the care that we take of all meters.

In our desire to satisfy our consumers, we do not go to quite the extremes, however, of the boy they tell about who was hired by a very conscientious and religious old dry goods merchant, who continually lectured his young employee on the value of quoting scriptures from the Bible to himself whenever the temptation to do wrong confronted him. He was very much annoyed and shocked one day when he witnessed the lad waiting on a lady who wanted a particular piece of material. He brought down a bolt from the shelf and placed it before her. She seemed quite satisfied until he said it was $1.00 a yard, to which she replied that she wanted something better than that. He carefully folded up the piece, reached under the counter and brought out a bolt of the identical thing and said it was a $1.50 a yard. "That's better," she said, "But haven't you something a little more expensive?" Not a bit fussed, the boy brought out the third and last bolt of the same cloth which he informed her was $2.50 a yard, and the very best they had. The lady paid for it and went away very content with the purchase. The door had hardly closed behind her when the proprietor descended in all his wrath on the boy. He ended his tirade by asking sarcastically just what passage of the scriptures he had been thinking of when he did such a crooked thing. The boy thought quickly and said "She was a stranger, so I took her in."

Returning to our office routine, all quarterly bills are rendered on regular United States Government post card forms. The actual billing is done with a standard book-keeping machine, which also is used for posting accounts when payments are made. Our ledgers are cards, 7 by 11 inches, unbound, and filed in trays which are kept on metal carts that can be easily wheeled about the office. Each card

has room enough to show the bills rendered and the payments made on one account for a five-year period. This has distinct advantages over the stub billing form, especially in small or medium sized companies. These advantages seem too obvious to enumerate, but where speed in billing is the essential thing and outweighs other considerations, then stub billing is the only method to use.

When one of our districts has just been billed, all the ledger cards are placed in the forepart of the trays in a section marked "Unpaid." As the payments are made and posted, these cards are removed and placed at the rear in a part marked "Paid." This enables us at any time to run off an adding machine tape of our outstanding accounts receivable, and greatly aids in striking a balance at the end of the month as well as speeding up the typing of delinquent notices because the clerk is not forced to handle paid accounts when all she is interested in, are those that remain unpaid.

Before continuing, there is another file which we have that may be worth mentioning. We call it the contract envelope file. The day that a customer signs an application for a new service, we place it in a heavy fibre envelope 10 inches long and 8 deep. An account, or route number, is printed on it, and from then on every important record in connection with that account is placed in it. As a result, if the curb box is buried and we wish to locate it 20 years from now we can do so by locating the original tapping order in the envelope. We thus have in one place easy to reach every permanent record of value, but we do not clutter it up with unimportant papers of only current use, such as turn ons, temporary turn offs, meter changes, complaint inspections and the like. These are recorded on the meter reading sheet, then kept in their respective files in route number order until their usefulness has been outlived, after which they are destroyed. This, again, is practical in only small or medium sized companies because it might be cumbersome from the standpoint of filing space required for it in a very large company's office.

All of our bills are rendered on the first of the month, and are due for payment no later than the fifteenth of the same month. We have no discount for prompt payment, but for our consumers convenience we have authorized neighborhood collectors who accept money up to the due date, but no later. Consequently, the majority of our bills are paid before this time because our main office in Buffalo is a considerable distance from most of the territory served, and people prefer traveling a few blocks to several miles or spending an extra amount

for a money order and postage, because very few of them have the luxury of a checking account. Which brings to mind a sign that appeared in a certain store shortly after the bank crisis, and is still there. It says, "To our Customers . . . Payments must be made in cash. We believe your check is good, but we do not trust the banks."

COLLECTION METHODS

In order to describe our collection methods, it will perhaps be clearer to choose just one district and follow it through chronologically, keeping in mind that the same things are being done in the other districts in successive months.

Taking District No. 1 as an example, we render our bills on the first business day of the month, and they are due for payment the fifteenth. By the thirtieth, practically 65 percent of them have been paid. About the middle of the second month, or 30 days after the due date, our first delinquent notices are mailed out in this territory. These are a reminder that the account remains unpaid and that we would like to have it settled without being forced to discontinue service. This printed form is designed to fit a window envelope and requires only a minimum of work by the clerk in typing it. Payments during this second month amount to about 12.5 percent of the original billing. At the beginning of the third month, a collector is sent into the territory with a list of the unpaid accounts. He makes a house to house canvass of all delinquent consumers, keeping in touch with the main office, and trying to collect these accounts without discontinuing service, a step which we do not take at this time. He has a task that I do not envy, for he is apt to be blamed for all the ills, real and imaginary, that have befallen a family in their lifetime. The other day, after an unprovoked and especially searing harangue from a woman, he asked her, when she ran out of breath, if she had anything else to say before he left. "Yes," she replied, "But I'm too much of a lady to say it."

When he was telling us of the incident, I could not help thinking of the same type woman who angrily confronted Lloyd George one day and said "Sir, if you were my husband I would feed you poison." To which he replied, "Lady, if I were your husband I would eat it."

But the man whom we have doing this work manages to keep his good nature and does a pretty good job. During this final month of the quarter we receive an additional 12.5 percent of the total billing, leaving about 10 percent unpaid at the end of the period.

DELINQUENT BILLS

By this time, the district has been billed again, but before our statements are sent out, a list is made of all accounts which show any arrears and total, including the current bill, $9.00 or more. These bills are then mailed out with the rest, but if they are not paid by the due date, a final delinquent notice is promptly sent by registered mail in compliance with the New York State Laws of 1934, Chapter 829, preparatory to discontinuing service for non-payment.

You are undoubtedly familiar with this amendment which requires water works corporations to send a registered notice to its customers 15 days before shutting off the water for the non-payment of a bill. The law at first required that, in addition to this, a notice must be posted in a conspicuous place on the premises, but in 1935 another amendment was passed making it unnecessary to post one and two family houses, but it still must be done for premises occupied by more than two families.

If the account remains unpaid at the end of the period mentioned, service is discontinued until the bill is paid in full, plus a $2.00 reinstallation fee to cover the cost of the efforts made to collect the account. It is thus over four months from the oldest item on a bill before we refuse a customer further credit. This may seem like a long time, but the State Law previously mentioned has slowed up our collection routine because the time, labor and money involved in applying it to all delinquent accounts instead of limiting it as we have, would involve spending more money than we now lose to the deadbeats who manage to keep one jump in front of the man with the turn-off key.

When we criticize this piece of legislation, we do not do so because we wish to go out and shut off the water of every consumer who does not pay his bill on the day it is due. As a matter of fact, we turned the water off for non-payment in fewer cases before this law was passed than since, because we now have a higher number of large unpaid bills. People do not like to have their service discontinued, and we do not like to be forced to do it, but formerly we could threaten to do so, and mean it, but now our threat is hollow unless it comes in a registered letter, and the people who are habitually delinquent know it. The legislation is actually beneficial to no one but those people who do not want to pay bills until they have exhausted every means of escaping them. In those deserving cases where people are just up against it, we are and always have been willing to meet them more

than halfway. We have never made it a practice to shut off water indiscriminately, and every case is carefully checked and the customer given every opportunity to come into the office and talk it over with us before any final action is taken. We are in constant touch with the welfare department and are thus able to be lenient where necessary, and apply pressure when that seems to be the only thing that will serve. It is a difficult problem at best and one that can never be solved by arbitrary or any other kind of legislation.

(Presented before the New York Section meeting, October 17, 1935.)

REPLACING WOOD WITH CAST IRON PIPE UNDER P. W. A. REGULATIONS

By Richard F. Wagner

(*Director, Public Works, Lynchburg, Va.*)

The Lynchburg Water System had its origin in the year 1799 when the local Fire Company was granted the privilege of digging several wells and erecting pumps "for the safety and convenience of the citizens." In 1811 John Lynch, the founder of the city, was authorized by the Council to convey water in wooden pipes from springs under his control, for which water he had the right to charge all who used it, the town reserving the right to use it free of cost for the purpose of extinguishing fires. In 1822 when the matter of water had become pressing the Council appointed a Committee to investigate the problem and to make recommendations. Such Committee's findings resulted in the establishment in 1829 of a pump house on the James River and of a reservoir at a sufficiently high elevation to serve the needs of the town. Most of the inhabitants were inimical to the whole project, claiming that water could not be carried that far up hill (about 280 feet) and on the day the pump was to be tested the leaders of the movement were advised to leave town. The works, however, were a great success, and it is recorded that when water gushed forth before the skeptical eyes of the throng gathered at the reservoir the crowd went wild with fervor and wonder.

In 1844 the office of Superintendent of Water Works was created by ordinance, carrying a salary of $600.00 per annum.

The James River for the better part of a century furnished an acceptable water. Though occasionally rains would detract from its physical appearance, owing to surface wash from cultivated lands finding its way into the stream, the supply was free from pollution. During the nineties, however, manufacturing plants were located above the City's intake. Industrial wastes from such plants were discharged into the river as the shortest and most economical method of disposal. This created an unsatisfactory supply and resulted finally in its abandonment except for auxiliary purposes. The new

supply from Pedlar River was inaugurated in 1907. This river is a typical mountain stream which is impounded to a storage of 600 million gallons by a concrete dam of the gravity type. The catchment area aggregates 33 square miles, the delivery conduit being 21 miles long.

SUPPLY MAIN

The original supply main was 30 inches in diameter, largely built of California red wood, except for cast iron pipe at river crossings and steel pipe at high pressure sections. About the year 1920 it became apparent that some portions of the wood pipe would require replacement in the near future because of rapid decay. For this reason the City in 1921 began to set aside from current earnings of the Water Department for future replacement, if and when necessary, the sum of $15,000.00 annually. Such reserve enabled the City to install over two miles of 36-inch C. I. pipe between the years 1927 and 1933.

P. W. A. PROJECT

During the fall of 1933 we investigated the desirability of accomplishing a major replacement under P. W. A. sponsorship. While unquestionably our supply main might have held for several years more, the increasing number of leaks in the wood pipe gave us a deal of concern. Then, too, the work would provide employment for men unable to secure work in private industry. The latter reason was probably the deciding factor which prompted the City to undertake the project. The work was approved by Washington during the latter part of 1933, the Government offering to purchase the City's bonds up to $450,000.00, the grant to be 30 percent of the cost of labor and materials going into the work.

LOCATION OF PROJECT

All of the pipe replaced in previous years was located near the dam, the plans for the P. W. A. project contemplating a further replacement of 48,000 linear feet, or all of the conduit north of the James River Crossing. There still remain at this time approximately 8½ miles of the original conduit, this being the section between the City and the James River. (An application for such replacement was turned down by Washington just recently, but we are hoping that it may be approved during the next term of Congress.)

DESIGN

We felt that some savings could be effected by using slightly lighter pipe with a higher tensile strength than provided by the A. W. W. A. specifications. We arrived at this conclusion since the conduit is operated with a minimum amount of water hammer and since the pipe is not exposed to impacts due to heavy traffic. We used what we were pleased to call Class "A" High Tensile Pipe with a nominal wall thickness of 0.80-inch up to 50 pounds head; Class "B" H. T. Pipe with a wall thickness of 0.92-inch for heads varying between 50 and 100 pounds per square inch. Above the latter head we used the conventional pipe thickness. In our design we analyzed, also, the effect of the relatively great weight of backfill in deep trench sections, and adopted the proper classification of pipe to these specific locations.

MANUFACTURE AND DELIVERY OF PIPE

All of our pipe was manufactured in the Radford Shops of the Lynchburg Foundry Company and delivered by rail at Lynchburg beginning the latter part of December, 1933, at the rate of twenty 12-foot joints per day, which program was systematically adhered to until the entire lot was delivered. The transportation problem between the Railway terminal and the site was handled by contract and great credit is due to the hauling firm which performed it. Anyone who is familiar with the topography of the country around Lynchburg will agree that the delivery of more than 4,000 tons of pipe was not exactly a sinecure. The contract stipulated that the pipe be delivered along the trench as directed by the Engineer. Every care was taken to so spot the pipe that little, if any, shifting had to be done later. In a few instances only, where grades exceeded 35 percent, was the pipe stored at a location other than opposite its final resting place.

CAMP FACILITIES

Since the average distance of the construction from the City was 17 miles, and since it was essential that citizens of Lynchburg be employed in its performances, camp facilities had to be provided. Because such camp had to be mobile, tents were used for housing the men and for mess halls, while a frame building of knockdown construction served as kitchen, storeroom and foreman's mess and recreation center. Camp was moved on five occasions and required two days for each set-up. The cost of operation, equipment

and transportation of men was $33,000.00 or $1.10 per man per day. The City made a charge of 90 cents and hence sustained a loss of approximately $6,000.00.

WORKING METHODS

All of the work was performed by force account, excepting the delivery of the pipe as stated before. Excavation and backfill were all done by hand, the 48,000 feet of trenching involving 51,000 yards of earth and 4,000 of rock. Drilling was performed with air drills and blasts were set off by means of a battery. The pipe was handled by a tractor equipped with a side-boom. In some few unusually steep places timber runways were placed in the trench and the pipe tramwayed to its final location from the crest of the hill. All joints were poured with leadite. Water from the leaking conduit paralleling the new installation at times slowed up the construction.

WORKING FORCES

All but a small percentage of the working crew, averaging 200 men, were secured through the National Reemployment Service. Since we worked our force 10 hours per day it was essential that men in good health be secured. Each man, therefore, underwent a thorough physical examination and approximately 15 percent of persons referred by the Employment Service were rejected because of some physical disability. Few men were lost because of sickness and our working output for this class of labor was comparatively high, substantial food and ample hours of sleep and rest contributing not a little to such results. Two shifts constituted the force, each working three days per week. Common labor was paid the usual 45 cents rate with semi-skilled labor correspondingly higher.

OUTPUT

The following tabulation indicates the efficiency of the labor available. In this connection it must be kept in mind that some men were inexperienced though willing; that the terrain was very rough, requiring much rehandling of the material, and that weather conditions at times were very unfavorable.

	man-hour
Earth excavation, per cu. yd	2.78
Rock excavation, per cu. yd	5.29
Backfilling, per cu. yd	0.88

	man-hour
Pipe laying, per linear foot	0.60
Drilling, per linear foot	0.13
Blasting, per hole	0.25

COSTS

The total cost of the project was $556,000.00 carrying a grant of $159,000.00, so that the net cost to the City was $397,000.00. Twenty-six percent of the total cost was expended in direct labor on the construction site. The total cost per foot of pipe installed was $11.58 or $8.27 net to City.

The project was completed within the twelve months period as planned. Much of the success of the undertaking must be attributed to the whole hearted support of the State Engineer and his organization. The United States Government in endeavoring to decrease the widespread unemployment existing at that time, not only accomplished this object, but contributed in a most concrete fashion in enabling the City to secure a capital installation of high economic value, which assures the supply of an ample amount of water for several generations to come.

(*Presented before the Virginia Section meeting, November 8, 1935.*)

WATER TANKS OF REINFORCED CONCRETE

By A. L. Hewett

(Superintendent, Water Department, Billings, Mont.)

In the year 1915, the City of Billings took over the privately owned water system and constructed a filtration plant with a capacity of 6 m.g.d. It also made extensive improvements to the pumping and distribution systems. This was accomplished from the proceeds of bond issues totalling $500,000.00.

In July, 1934, the City found itself in the fortunate position of being able to pay off the balance of this bonded debt, as well as having made the necessary improvements and additions to its distribution system and acquiring a substantial surplus. All this was paid for from the earnings of the water department from the proceeds of rates comparable with those of the average municipally owned plants in the country. It was, however, considered desirable to enlarge and improve the plant in order to be in a position to take care of the future requirements, the plant being taxed from 25 to 30 percent above its rated capacity during the summer months. Black & Veatch, Consulting Engineers of Kansas City, were, therefore, retained to make an extensive survey and report on our water system, together with recommendations for improvements.

Their report outlined a program of improvements covering a five-year period, the cost of such improvements to be paid from the earnings of the water department. This report was adopted by the City Council, and this firm of engineers was retained to prepare plans and specifications for these improvements. The first year program included the construction of additional filter units to increase the filter capacity from 6 to 10 m.g.d. Work was started on the construction of this improvement late in October, 1934, and the concrete work has been nearly completed. Equipment has been purchased which will be installed in time so that advantage may be taken of the increased filter capacity for the heavy demand put upon the system during the mid-summer months.

CONCRETE CONSTRUCTION

Some information relative to the specification requirements and construction methods adopted in connection with the building of the concrete for these filter units will probably be of interest to the members of our Association.

Briefly, the specifications required a cement-water ratio concrete, proportions being 5.4 gallons of water per sack of cement, seeking a 28-day strength of 3,750 pounds per square inch. This specification was rigidly adhered to, and results of breaks in the sample specimens show an average compressive strength in excess of 4,000 pounds per square inch. This high strength concrete was required more to insure water-tight concrete than for strength, inasmuch as the bottom of this structure is from 12 to 14 feet below where we may expect the ground water level to be at times.

Every precaution was taken to insure a concrete of uniform strength and density; all aggregates were weighed in batch weighers, one being provided for each fine and coarse aggregate; frequent checks were made on the moisture contents of the aggregates and corresponding adjustments made to the amount of water added to a batch; a concrete vibrator of the "spade" type was used throughout the construction. The type of construction necessitated a high percentage of reinforcing and our primary idea in using the vibrator was to eliminate a large amount of the labor required to satisfactorily tamp this concrete. However, we soon found that this vibrator, properly used, also resulted in our being able to use more coarse aggregate and being able to handle a much harsher concrete with an absence of "honey-combs"; this, of course, meant a resultant use of less cement and a denser, more water-tight concrete.

Pouring of the concrete in this structure during the winter months necessitated the protection of the concrete from freezing and the adoption of some method to insure that the concrete would not be damaged from this element. A large steam boiler was installed on the work, which provided steam for the heating of the water and aggregates in the mix, and for curing the concrete after it was poured. We were very fortunate, however, in not having any extremely cold weather to contend with during the construction.

WASH WATER TANK

In our present plant, the water for washing the filters is furnished by special pumps for this purpose. To obviate the necessity for these

pumps in the present plant, as well as to provide the new units with wash water, it was recommended by the engineers that a 75,000 gallon elevated wash water tank be constructed, thus eliminating the wash water pumps together with their operating expense and furnishing a more acceptable and satisfactory means of securing the necessary wash water for our filters.

After much thought and discussion, it was decided to build this tank of reinforced concrete. Inasmuch as this water tank has features of construction which differ quite materially from the ordinary concrete construction, we have made this the main topic of our paper, as we feel that our members will find it of special interest, and the knowledge of which will provide a means of overcoming some of the difficulties they have to contend with. These special features have a very important bearing upon the construction of structures for the storage of water.

The members of the New England Water Works Association took an early interest in the construction of water tanks of concrete. The unsightly appearance, the expensive upkeep and frequent periods of idleness for painting and repair of steel tanks made a more durable method of construction desirable. From 1900 to 1912 many important tanks and standpipes of reinforced concrete were built in the New England States. In place of securing permanent and attractive structures, quite the opposite was the result. A review of the troubles encountered may be found in the Journal of the New England Water Works Association for 1915.

Some of the more important structures built were the following:

LOCATION	DIAMETER	HEIGHT	YEAR
	feet	*feet*	
Attleboro, Mass.	50	102	1904
Waltham, Mass.	100	37	1906
Manchester, Mass.	50	72	1909
Westerly, R. I.	40	70	1910
Lexington, Mass.	30	104	1912

The history and behavior of these structures are reviewed in the Journal referred to, but briefly the concrete cracked, resulting in leaks, ugly discolorations and ultimate disintegration. Many expensive attempts were made to repair them without success.

The trouble with reinforced concrete structures of the character is

due very largely to the shrinkage of the concrete from the loss of its water content. In this respect concrete is much like timber. If we should build a wood stave tank by placing the bands through the center of the staves without means for adjustment, we would have an unsatisfactory structure which would leak due to the shrinkage of the staves and the elongation of the hoops under pressure. A reinforced concrete tank, built by placing the steel in the concrete, is similar to such structures, and will give trouble in the same manner. When we build a wood stave tank we place the steel bands outside the staves with means of adjustment, and we then tighten these bands until shrinkage of the wood staves and the elongation of the steel due to internal water pressure will only result in decreasing the compression in the staves without allowing them to separate and allow leakage.

While engineers have known for many years that concrete shrank during its period of setting, the exact nature and magnitude of the shrinkage was not well known until about 1915 when exhaustive experiments by Mr. T. R. McMillan of the University of Minnesota gave the desired information. This was followed in 1921 by further experiments in the University of Illinois. This information indicates that the ultimate shrinkage of concrete is 0.00065 of its dimension. We may get a better idea of this by saying it is more than $\frac{3}{4}$-inch for each one hundred feet. The difficulty of building a structure of concrete to hold water without due consideration to this important feature is apparent.

Engineers informed in the design of reinforced concrete structures such as buildings and bridges felt themselves competent to design structures of reinforced concrete for the storage of water, since they knew how to proportion the parts for the known stresses. But the concrete in bridges and buildings may and does crack without affecting the strength or durability of the structure, while in a structure subjected to water pressure, the cracking of the concrete is of vital importance.

The design and construction of reinforced concrete reservoirs and tanks made but little progress until 1920, when Mr. William S. Hewett of Minneapolis and now of Chicago, my relative and business associate in the construction business for many years, invented what he styles the "Hewett System" of construction. Briefly this consists of leaving the steel bands out of the wall of the structure and placing them around the walls of the tank when completed and with means of adjustment exactly as we learned was the logical way to build wood

stave tanks. After these bands are adjusted they are buried by an additional wall of concrete. This additional concrete preserves the rods from corrosion, gives additional bearing and bonds the bands to the wall. This makes it impossible for the bands to come loose by pulling into the concrete as happens on the wood stave tank. Here the rod has a bearing along a line when placed, but has a bearing equal to its diameter after being buried, which effectively prevents pulling into the concrete.

Many engineers design structures for the storage of water and consider only, in the analysis of the design, the stresses due to water pressure. They consider shrinkage and temperature stresses as minor matters, and contend that stresses due to such forces are taken care of by the reinforcing. But shrinkage stresses in concrete increase with the percentage of steel and the amount of steel that can safely be placed in the concrete is limited. In the experiments in the University of Illinois (1921) the investigator concluded that 2 percent of steel placed in the concrete would cause the concrete to crack without the application of other forces. In some of the Massachusetts structures, the percentage of steel exceeded 5½ percent.

Temperature stresses are a vital element in the design of reinforced concrete structures for the storage of water. The coefficient of linear expansion for concrete and steel is about the same and for a change of 100 degrees F., the coefficient is about the same as the coefficient of ultimate shrinkage of concrete, or about ¾-inch per 100 feet. When we consider that shrinkage and temperatures may act together and that, in this climate, we have extreme changes of over 140 degrees resulting in a total change of length when combined with shrinkage of more than 1¾-inch for each 100 feet, we should be surprised, not that our large reservoirs leak, but that anyone should expect them to hold water for any considerable length of time.

The trouble due to shrinkage and temperature is slow in developing. Often a structure when completed seems entirely satisfactory, only to develop a bad condition after a few years. Note what Mr. B. Brewer, City Engineer of Waltham, Mass., says of the Waltham Tank.[1]

> The result has been this. The first year or two there was very little seepage; some at joints, and a good deal of efflorescense, and some stalactite formation on the outside of the wall. After being in use for a year or two the seepage began to increase considerably so that when the tank was full a considerable portion of it has been and is quite wet.

[1] Jour. N. E. W. W. A., 1915, p. 189.

Again note what Raymond C. Allen, City Engineer, Manchester, Massachusetts, says of the Manchester Tank.

> I have noticed, first, that the greatest amount of seepage has occurred on the south and west sides, where the expansion seems to be most unequal. I have also noticed that as from time to time the standpipe is lowered entirely and filled again, the successive filling having produced new points of seepage. At all events, they act a little differently and are cumulative in their effect. Thus, each time a standpipe is emptied and filled, I believe that a slight increase in seepage for a time, at least, takes place.

It appears that shrinkage and temperature are constantly acting together to destroy such structures. Where no logical method of contending with these forces is provided, we may expect our troubles will increase with time.

The troubles due to temperature are not entirely due to seasonal changes of temperature, but also the fact that at times there is a wide difference in temperature between the water inside the tank and the air outside. If the water is cold it tends to shorten the wall and if the air is warm that tends to lengthen the wall. Out of this conflict we get tension on the inner or cooler surface and compression on the outer and warmer surface. These stresses are the same in all directions and cannot be overcome by reinforcing since steel has the same coefficient of expansion. By the proper use of steel bars we may limit the cracks. Trouble from these stresses is evidenced by horizontal cracks on the weak horizontal planes of the concrete. Concrete as a rule is placed in horizontal layers, but from one to several hours may elapse between the placing of adjacent layers. As soon as concrete is deposited it begins to throw off its surplus water as the chemical action takes place. This surplus water comes to the top and brings with it a greasy film which will make it impossible to secure a perfect bond if any material time elapses between layers. In most concrete there are many of these weak horizontal planes which result in the concrete cracking in horizontal cracks of "lifts." The conditions one day open the crack on the inner surface and again, under a change of conditions, the wall is opened on the outer surface. These cracks do not close but gradually get worse until the conditions are serious. The walls of our Billings tank are built in vertical sections. In this feature we again consider the lesson of the wood stave tank. The tightened bands provide for taking the horizontal component of the temperature stresses while the section built without horizontal joints is able to resist the vertical component of these forces. To

insure this the wall is reinforced by vertical bars whose area is limited to 0.5 percent.

In most structures built for the storage of water, the horizontal bars are large and numerous, making the proper puddling of the concrete practically impossible. In the system which we have adopted in the construction of our tank, the horizontal bars are omitted, resulting in a much better class of concrete.

The first tank to be built under this method was constructed at Barnum, Minnesota, near Duluth, in the year 1921. Fourteen Minnesota winters have left the structure exactly as when built. This method has been adopted in the construction of numerous tanks and reservoirs since that time, throughout the United States. They have been built in all climates, below sea level in the hot Imperial Valley of California and in the coldest sections of the Northern States, with capacities ranging up to 3,000,000 gallons. A five million gallon unit is to be built near San Francisco this year and bids will be taken on an elevated tank of three to four million gallon capacity.

These structures are given attractive features at very little additional expense. The inner tank is built in the usual manner and the architectural features are considered in the outer wall. One of the most striking examples is a standpipe in Washburn Park, Minneapolis, Minnesota. When it was suggested that we build a concrete tank at Billings, our consulting engineers, Black & Veatch, readily agreed that the "Hewett System" was the proper method of design. They have, in their design, made the architectural features to conform to the adjoining buildings; while it is not an elaborate design, we believe the architectural effect will be decidedly pleasing.

We have devoted the major portion of our paper to this method of reinforced concrete construction for two reasons, viz: (1) it is a comparatively new method and its apparent advantages over the old method have been set forth in order to provide opportunity for discussion of its apparent advantages and disadvantages. (2) the application of this method of construction would, we believe, result in economical, efficient and water-tight structures. Every one of our members who has anything to do with the design or operation of structures of this type, realizes the damage and expense caused by leaky reservoirs or tanks, from loss of water as well as the actual damage done to the structure and the consequent expense of repairs. For these reasons, we feel that anything that can be presented to you, to assist in solving such problems, may prove of benefit.

(*Presented before the Montana Section meeting, April 13, 1935.*)

THE SMALL TOWN WATER SYSTEM, ITS PROBLEMS AND PERPLEXITIES

By M. H. McGuire

(*General Manager, Water and Light Department, McMinnville, Ore.*)

McMinnville, my home town, is situated in the Williamette Valley about forty miles southwest of Portland. Some fifteen years ago I was given the management of the municipal electric power plant there, and a few years later management of the City's waterworks was added to my duties. Just as parents sometimes appear to lavish the greatest affection on an erring child, I must confess a great fondness for the "baby" which has caused me a good many wakeful hours. I am happy to say that it has greatly improved as it has grown older and with the early completion of a "finishing course" prescribed by the able Commission under whose direction I work, it promises to become chief of the City's many prideful possessions.

That an abundant supply of palatable and safely potable water is vitally necessary to health and general welfare of the inhabitants of every city and town, great or small, has been pointed out by able speakers at sessions of this Association and elsewhere. The methods and agencies which may be employed to improve the quality of public water supplies should be familiar to all. The splendid results achieved in prevention of water borne diseases as revealed in statistics compiled by State Boards of Health, should convince everyone in authority of the reasonableness of the exacting requirements laid down by those Boards. But there remain certain perplexing problems peculiar to the small town system and I shall make brief comments here on such of these as have come under my personal observation.

PLANNING AND ENGINEERING

If your City considers establishing a system of waterworks, or wishes to re-build an obsolete or inadequate system which it owns or expects to acquire so as to meet its present and future needs, it is very important that a definite working plan be adopted as the first

step. Based on careful study of all available information, providing for future development and proceeding by orderly steps toward its final objective, such a plan may be made so comprehensive, may so completely cover every phase of development, that it will be necessary to deviate from it in no important particular during construction. If yours is a gravity system, thought should be given to protection of the source of supply against removal of timber, and contamination. The plan should be formally adopted by the Council or Board having the project in charge and a file containing complete details of the plan and all preliminary studies should be established. If graphs and other condensed records obtained from operation are regularly added to this file, other City Officials will be enabled to build for future requirements with understanding and economy on the firm foundation which your intelligent planning has provided. At the very beginning consult your State Board of Health and your State Water Engineer. They will furnish information and advice of great value without cost. Secure their approval of your completed plans and invite them to inspect the work under construction. It is economy to employ the best engineer whose services are available to you, one of recognized ability and large experience, preferably one who specializes as a water engineer. Usually his fee will be no greater than that asked by an engineer who lacks both the experience and the special knowledge needed in successfully planning your project, but the difference in results will be noteworthy and will stand out with increasing distinctness as the years pass.

FINANCING: BOND ISSUES AND REFUNDERS

From this source spring many problems of the first magnitude, the solution of which has robbed many a brain-weary executive of much needed rest. The average small city in its eagerness to wear big city shoes has often been led into carrying on numerous ambitious projects which a few years ago would have been considered entirely beyond its reach, with the result that it soon approaches the limit of indebtedness fixed by constitutions and charters. Bonds issued for water plants which have outlived their useful lives often remain outstanding with no definite plan in effect for their retirement. In some instances, perhaps, the original issue has been refunded and other subsequent issues have been floated, all without making provision for payment. There is just one way to meet this situation; that is, to start now to do what should have been done in the beginning. When nearly all the

receipts from a water plant must be applied to the payment of accrued interest, it is obvious that payment of indebtedness and provision for repairs, extensions and replacements will be out of the question and rates will likely continue to be high. New issues and refunding issues should be arranged with serial maturities, and if revenues are not sufficient to meet them, then the City authorities and the inhabitants should have the courage to provide funds by taxation to take care of the earliest series, and continue this program until the project becomes self sustaining. The injustice of the old idea of piling up indebtedness for posterity to pay is just now becoming apparent with painful vividness, and I suspect that some of the buck-passers might be in for a bad half hour occasionally if they had not gone where posterity hesitates to follow. On the whole I think the best thing I can suggest toward the solution of these problems is for those who inherited them to buck up and tackle them in dead earnest. There must be some way out.

RATES

When a plant is ready to give service the all important question of what to charge for that service must be answered, and if it is not answered right it may be a long time before the plant's operations will give the satisfaction which its sponsors have a right to expect. Charters sometimes provide that the Council or the Board responsible for the management of the plant *shall annually prescribe a rate for water service sufficient to meet all the expenses of operation, depreciation, interest on its funded debt, and that the sum to be collected from plant revenues may also include a sum to be used in accumulation of a fund to anticipate bond maturities, and may also include a further sum for plant extensions and improvements.* Now, would it not be just perfectly lovely if we could do it just like that and get away with it? But, alas, human nature being what it is, the average executive not being made wholly of iron, and the interference with which he must cope being something to talk about, he does not always chalk up a perfect score.

The principle of fixing the rates to accomplish these purposes should not, however, be given up. The minimum charge should not be so high as to take water out of the class of necessity and make it a luxury, and the rates should be so adjusted that one class of consumers will not be preferred over another, particularly if the system is being operated without meters. But it is my belief that no system, large

or small, can long be operated with any degree of success unless the service is metered. All service rendered the City itself in all its departments should be paid for at reasonable rates. It is too much to expect a plant to be self sustaining and self liquidating if such service as this is rendered without charge. Moreover, free service to the City results indirectly in an unauthorized and unjustifiable tax on the rate payers for the benefit of property owners, in that it reduces the amount necessary to be raised by City tax.

After the system has been in operation for a few years the management is sure to be besieged by well intentioned consumers who want to know why water is allowed to run over the dam and do nobody any good instead of supplying a larger amount at a lower cost to the consumers. No matter how much time is spent in detailed explanations of the reasons, a skeptical light usually shines from the complainer's eyes as he, or more often she, leaves the office and you know perfectly well what the consumer is thinking of you. It will also be suggested by public spirited citizens that, if the Board would offer free service to all who would come and establish manufacturing plants in our beautiful little city, the population might be doubled in just a very short time and between these two propositions lie a multitude of others, often presented by silver tongued orators who can almost make you believe that a grindstone is a square rock with a round hole in the center.

COLLECTIONS AND DELINQUENTS

In the small town where you know everybody and everybody knows you, the manager of the waterworks must be a combination of Czar, Diplomat and tear-shedding politician if in these times he holds the delinquent list to decent proportions. Water service is a commodity which is always sold on credit and the same principles which apply to credit extension in other lines of business, apply with equal force here. True, consumers not owning taxable property in the City may and should be required to maintain a deposit to offset the monthly bill, but even then extreme care and constant watchfulness are necessary because there are so many plausible reasons why just a few days' delay will be a great accommodation, and when the bill is larger than the deposit, they have you going. You have the advantage of knowing when it would be a real kindness to grant an extension of time to some hard-pressed consumer, possibly a widow with hungry mouths to feed, and when you should think faster than the customer who

merely wants to use the money for some other purpose, but the problem is so to handle each customer that he will not spend the time between collection days in thinking up new stories to convince you that decreasing payments each month will not pile up a bill which ultimately you will have to charge off. Resort to discontinuance of service is seldom necessary, but when it becomes so there should be no hesitation because, if a customer has been told that the water will be shut off and it is not done, the news will travel fast and you may have to shut off a lot of them in order to convince them you are in earnest. Before water is shut off the Manager should be sure that it will stay shut off until the bill is paid and that no tender-hearted Mayor or Board member will interfere. During the depression years many Managers have collected bills by receiving cord wood, poultry or other produce from delinquent consumers, which they generally sold to members of the organization, sometimes at a loss. Others have employed delinquent customers on work which could have been deferred, but all these things are justifiable because the consumer's self respect has been preserved. He has paid in the only way he could pay.

METERS AND MAINTENANCE

Metered service is essential to the success of any water system. The reasons for this conclusion are so well understood that it is useless to repeat them here. The only question upon which an argument might be based is as to whether or not investment in meters and the cost of reading and billing is more than offset by the added revenue produced. The saving of water is of more importance in some systems than in others. The result of my experience in selling metered service convinces me that meters do this consistently particularly if the meters are kept in adjustment and worn parts of old meters are replaced with new so as to keep the meters working properly instead of buying new ones. The saving possible here is considerable. To simplify this work I would confine purchases to one type of meter, with which I would require employees of the system to familiarize themselves, 2 would maintain a stock of parts and repair, adjust and test all the meters in the system at odd times when the work is slack. Perhaps meter manufacturers may not agree with my conclusions with respect to buying only one type of meter, for, of course, there are several which give equally good service, but my reason is that an

employee of a water system in a country town could not well undertake to master the details of several types, and the stock to be carried can be kept at a minimum by adoption of a single type.

There are numerous and improved new types of equipment which increase many fold the efficiency of the small force regularly maintained by a small water system. The question of whether or not it is desirable to make an investment in such apparatus is one to be settled by the management of each system, but in general it may be said that if the money is available a saving will always result from such an investment. To mention one instance, we own in our department a first class oxy-acetylene welding outfit and have developed experts in its use within the ranks of our limited force so that we are able to weld and lay steel pipe rapidly, efficiently and economically without increasing our payroll.

ADVERTISING

The question has been asked, "Why advertise a prime necessity upon the distribution of which you have an absolute monopoly?" In discussions the weight of opinion seems generally to have been against the practice, although there is still, here and there, an advocate of the benefits which may be derived from advertising and the giving of prizes for well maintained gardens and grounds in an effort to stimulate consumption of water for irrigation during the growing season. Sales during this period are also stimulated considerably in some systems by a substantial reduction in rates during this period. This is only possible, of course, in systems where there is a surplus of water.

In a municipally owned system, however, where every citizen is in effect a stockholder, it may be desirable to use advertising as a medium to acquaint the citizen with the nature and extent of their investment in the water system and to instill what the automobile people used to call "Pride of Ownership."

It has been found in the case of Mutual Fire Insurance Companies that a statement to policyholders that insurance is furnished them at cost coupled with an appeal to them to use their influence against maintenance of fire hazards, has always resulted in a noticeable decrease in losses. May not a similar appeal to citizens who own a water system result in better enforcement of regulations and limitation of waste? I offer the thought without any conclusion.

CONCLUSION

In spite of the problems and perplexities inseparable from the administration of any water system, I know of no business, the management of which inspires more enthusiasm and determination to surmount all difficulties and more satisfaction in accomplishment than this one. Especially is this the case where the Supervising Board has a deep understanding of the problems involved and a correspondingly deep appreciation for difficulties surmounted, and where employees are loyal in the highest degree and are a never failing source of inspiration to their Chief. I am truly thankful that my lot has been cast with such associates.

If it will not weary you I should like to say a few words here concerning the water system at McMinnville with which I am connected. This system, in my opinion, approaches very closely to the ideal of what a small-town water system should be. It is unique, so far as I know, in the absolute ownership of 80 percent of the 5000 acres included in the drainage area of the source of supply. It is purely a gravity system and no pumping is required at any stage of distribution. The supply is abundant, storing upwards of 500 acre feet for use in low water periods. It is capable of expansion at the same reservoir site so as to increase its capacity many times. The water is clear, sparkling, palatable, requires no filtration and is treated for the destruction of bacterial life as a precaution rather than as a necessity. The system is self supporting and at the rate of retirement now being maintained will be debt free in about fifteen years.

(*Presented before the Pacific Northwest Section meeting, May 17, 1935.*)

THE WATER FILTRATION PLANT OF HAMILTON, ONTARIO

By W. L. McFaul

(City Engineer and Manager of Waterworks, Hamilton, Ontario, Can.)

The filtration plant of the Hamilton Water Works was put into operation in March, 1933, so that the citizens of Hamilton have been provided with filtered water for two years. The experience of the actual operation of the plant proves conclusively that there are many variable conditions which have to be met in designing a filtration plant to treat Lake Ontario water. The plant itself is located on the Beach Water Works property, and the following works are included: low-lift pumping station, chemical building and mixing chambers, settling basins, filter building, wash water tower, offices and laboratory, filtered water reservoir and connecting conduits.

The City Council, early in 1930, decided to construct the filter plant after complaints from the citizens, and particularly manufacturers, concerning turbidity. The writer was instructed by the City Council to proceed with the design and construction of the plant.

In order to provide an adequate raw-water supply, a new 60-inch intake and conduit were constructed in 1927. The old 48-inch intake and conduit were connected with a 48-inch concrete conduit, and the old settling basin on the shore of the lake was abandoned. A screen house with revolving screens on the 60- and 48-inch conduits were provided, as well as a chlorine house and conduit connecting to the existing high lift station. Provision was made in the connecting conduit between the screen house and high lift station for two 60-inch diameter conduits to connect with the proposed low lift pumping station.

The plans and specifications were prepared and tenders let for the first part of the work in January, 1931, and all the works were completed in March, 1933.

LOW LIFT PUMPING STATION

The low lift pumping station is constructed of reinforced concrete substructure and red pressed brick with Queenston stone trim for

the superstructure. The crane is carried on steel frame construction. The roof steel is encased in concrete, and the slabs consist of precast haydite and are covered with 2 inches of fibre board and a built up tar and gravel roof. The main pump building is 35 feet by 100 feet. The electrical equipment is housed in an addition on the west side 12 feet by 80 feet. In the centre on the east side of the main pump room is an addition 25 feet by 45 feet which contains the main entrance, offices, and pump room for dewatering pumps.

In the large pump room four horizontal centrifugal pumps are installed. These pumps were supplied by Babcock-Wilcox and Goldie-McCulloch Ltd. of Galt, and are 5½, 11, 22 and 33 m.g.d. capacity, respectively. The suction well is a reinforced concrete box running the length of the pump house, and is connected to the two 60-inch conduits which supply the raw water. The pumps are located on the pump room floor above the suction well. The suction pipes connect to the bottom of the pumps and are provided with gate valves. The side discharge pipes from the pumps, after passing through the floor of the pump room, are equipped with tilting disc non-slamming check valves and gate valves. The check valves were supplied by the Dominion Engineering Works Ltd.

The pumps are driven by line start 2200 volt constant speed motors. Push button stations on individual control pedestals located in the main pump room control the priming of the pumps and operation of the pump motors. A nine cubicle switching structure was installed in the westerly addition and contains the necessary measuring instruments, circuit breakers, switches, etc. These motors, the control pedestals and the nine cubicle metal clad switching structure were manufactured by the Canadian Westinghouse Co., Ltd., and installed by the Culley Electric Co.

The welded steel pipe discharge header located in the basement is connected at the north wall of the building to a 60-inch diameter reinforced concrete discharge line, which conveys the water to the chemical building. Here the pumpage is recorded by a Guest and Chrimes Ltd. 60-inch Venturi meter, supplied by the W. J. Westaway Co.

CHEMICAL BUILDING AND MIXING CHAMBERS

After passing through the Venturi meter the water is delivered to two flash-mixing units located in the chemical building. This building is constructed of reinforced concrete substructure, and red pressed brick and stone trim superstructure. The roof is a reinforced concrete joist and slab structure poured in place.

A railway siding is provided at the north end of the Chemical Building, and alum is received in bags in box cars, and the bags are elevated in the alum storage by freight elevator. Two Omega dry feed machines are placed with hoppers below the top storage floor and the alum is placed in the hoppers and fed into tanks below the machines. The dissolved alum is pumped to the flash-mixing units where it is mixed with the raw water. The flash-mixers were supplied by the Turbo-Mixer Corporation, and each consists of two impellers on a vertical shaft which is motor driven and revolves in the water chamber.

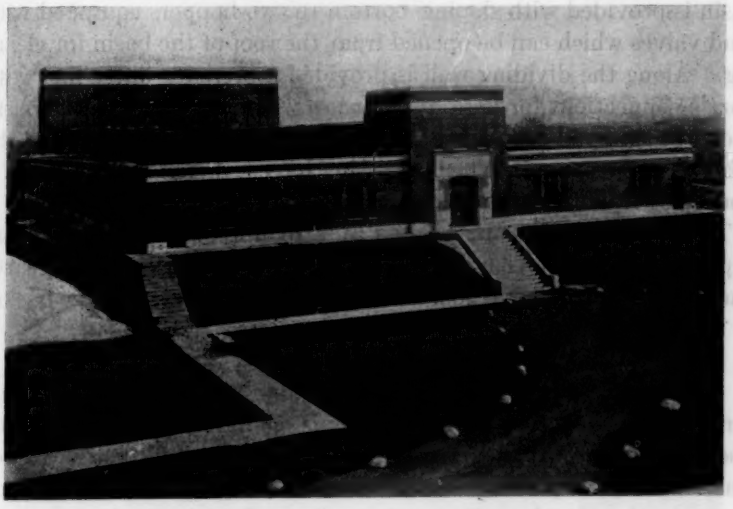


FIG. 1. EXTERIOR VIEW OF FILTER PLANT

The treated water then passes along a channel to the mixing chambers which consist of four units each 30 feet wide by 60 feet long by 17 feet deep. The water enters the mixing chamber through slots and discharges over a weir. Each mixing chamber is equipped with two Dorr Company flocculators. Each flocculator consists of a shaft running the length of the tank with four staggered wooden paddles 14 feet in diameter. Each shaft is driven at the centre by motor and variable speed gear box through sprockets and chains. The paddles revolve from the top towards the centre of the tank. The speed may be varied from 1 to 3 r.p.m. The detention period in the tank is 30 minutes. By means of the chemical treatment and

mixing, we are able to produce a floc sufficiently stable so that it is readily settled out in the settling basins.

SETTLING BASINS

There are two settling basins, each 111 feet wide by 264 feet long by about 14 feet 6 inches deep. The water is carried from the mixing chambers in a collecting conduit, and distributed to the settling basins through slots, and baffles are provided at the entrance end between the end wall and first columns. The water flows from the entrance end to the outlet end and is discharged over a weir and collected in a conduit and conveyed to the filter building. Each settling basin is provided with sloping bottom to two hoppers equipped with mud valves which can be opened from the roof of the basin for cleaning. Along the dividing wall is provided a pressure water line with nozzle connections for hose lines for use when cleaning. Each basin is controlled by sluice valves which permit it to be cleaned while the other is in operation. The two basins have a designed capacity of 3 hours detention at 40 m.g.d. rate.

The settling basins are covered by a reinforced concrete roof slab supported on columns and the roof slab is covered with gravel, earth and sod. Control valve houses are provided at each end of the basins.

FILTER BUILDING AND FILTERS

In the filter building twelve filter units are provided which are arranged in two rows separated by a common pipe gallery and operating floor. At the outside end of each row of filters the settled water conduit is placed and underneath this the waste water sewer. Hung on the outside wall is a pipe gallery, the top of which forms a walk on three sides of the building.

The operating floor is level with the top of the walks provided on the filter walls and the central portion over the rate controller gallery is omitted. Light and air are thus provided for inspection of the rate controllers and valves. Each filter is controlled by a 20-inch Simplex direct acting type rate-of-flow controller. In the central gallery the filtered water tunnel and the cast iron wash water line are located. A tiled walkway is provided on top of the wash water line.

The filter sand area of each filter is 1,438 square feet and the rated capacity is 3.33 m.g.d. The filter box is divided by a central drainage channel thus making each half filter 16 feet wide by 45 feet long. In the centre of each half filter and beneath the floor, a manifold

distributor extends the length of the box and it is 42 inches wide by 30 inches high. The manifold is provided with a cast iron top and bosses into which the cast iron pipes are leaded. These pipes comprise the underdrainage system and are 4 inches in diameter and spaced 12 inches apart with $\frac{9}{16}$-inch diameter orifices on 6-inch center. The two manifolds are joined at the gallery end, and are connected by cast iron piping to the rate controller, and filtered wash water line. Operation and back washing are facilitated by two hydraulically operated gate valves in the central gallery and two hydrau-

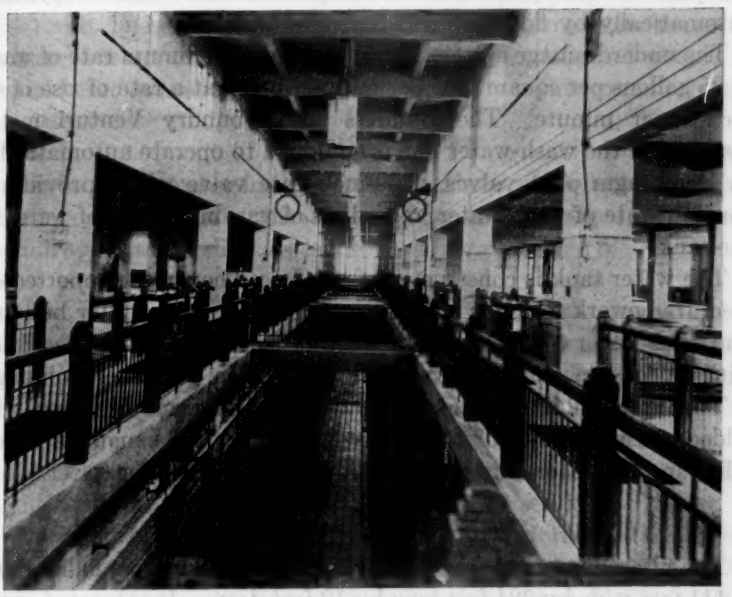


FIG. 2. INTERIOR VIEW OF FILTER PLANT

lically operated sluice valves on the inlet and sewer outlet of each filter. The underdrainage system is covered with gravel 20 inches deep in five layers from $2\frac{1}{2}$ inches to 10 mesh, and on top of this gravel 30 inches of sand from Cape Cod, Mass. are placed. This sand has an effective size of 0.50 mm. and a uniformity coefficient of 1.6. The wash water troughs were constructed of reinforced concrete cast in place. The edges of the troughs were carefully ground to level and the accuracy tested by water level.

A control table for each filter is provided and the top and trim-

mings of the table are marble. Removable front and rear panels give access to the control piping. On each table a Simplex loss of head gauge and a rate of flow gauge are combined in one unit, and also the four valve control mechanisms.

THE WASH-WATER TOWER

At the east end of the filter building is the wash water tower with a capacity of 168,000 gallons. Two Smart-Turner wash water pumps of 2,000 g.p.m. capacity supply water to the tower. Each of these pumps is driven by 60 H.P. Westinghouse motor and is operated automatically by float control.

The underdrainage system is designed for a maximum rate of wash of 25 gallons per square foot per minute, being at a rate of rise of 42 inches per minute. The Builders Iron Foundry Venturi meter installed in the wash-water line is equipped to operate automatically by diaphragm pilot valves, a 30-inch gate valve which provides a constant rate of wash water, regardless of varying height of water in the tank.

The water tank is constructed of reinforced concrete supported on steel framework which is encased in brickwork. The filter building and wash water tower are constructed of Hamilton red pressed brick with Queenston stone trim in conformity with the other buildings.

The filter building is laid out so that 12 additional filters may be added on the east side of the wash water tower. The main entrance, office and laboratory are provided under the wash water tower.

FILTERED WATER RESERVOIR

The filtered water reservoir has a capacity of 2 million gallons and is 111 feet wide by 284 feet long by 12 feet deep. It is located south of the filter building, and connected to it by a filtered water conduit. It is constructed of reinforced concrete and covered with a flat slab roof which in turn is covered with earth and sod for protection. On the inlet end of the reservoir a small brick building houses a Wallace and Tiernan Vacuum Chlorine Machine.

CONNECTING CONDUITS

From the west end of the reservoir the filtered water is carried in conduit in a northerly direction to the south end of the low lift pumping station, and then westerly to connect to the end of the high lift pump suction conduit. Here the water is pumped to the City

through the distribution and reservoir system. Provision has been made for the construction of a future high lift station at the south end of the low lift station.

ELECTRICAL EQUIPMENT

Two 13,200 volt power lines, an aerial and underground line provide power to three 1000 K.V.A. transformers through electrically operated switching gear and circuit breakers. A spare transformer is provided to be cut in, in place of any one failing in operation. This station is located north of the old steam pumping station. Two 2200 volt lines were carried from this transformer station to the existing bus system in the high lift pumping station, and also to the switchboard in the low lift pumping station by underground conduits.

SEWERAGE SYSTEM

The sanitary sewage from the plant and houses is collected, pumped, and treated in a septic tank. The effluent is discharged to the main outfall sewer which discharges into Lottridge Creek.

HEATING PLANT AND PIPE TUNNELS

Use is made of the existing boilers in the Osborne Killey pumping station for heating the buildings comprising the filtration plant, in addition to the high lift station and other buildings. Steam lines are carried from the boilers in conduit to the low lift pumping station and in tunnel to the chemical and filter bulldings, respectively. These tunnels also provide for power cables and other services.

The cost of the plant, including the transformer station, was $1,350,000.00 including loss on sale of debentures of $130,000.00. This leaves the construction and engineering cost at $1,220,000.00.

In the operation of the plant it has been found desirable to treat the water with ammonia and chlorine before it reaches the low lift pumps. The ammonia is applied ahead of the chlorine, to an amount about 25 percent of the required chlorine. Most of the time residual chlorine is maintained through the plant and no post chlorination is required. Owing to the slight turbidity of the water a great portion of the time the alum dosage is reduced to intermittent dosing, and often eliminated entirely. The mixing tanks are cleaned once in two months. The settling tanks are cleaned once a month in winter, and once in two weeks commencing in June and continuing to October.

The water temperature rises from 35°F. in winter to 70°F. in summer. In back washing the filters the rate of wash is determined by the temperature of the water and sufficient rate of rise used to expand the sand 50 percent. We also limit the length of runs to 48 hours between backwashes, or a loss of head of 8 feet. During the summer algae frequently shorten our runs to 30 and even 20 hours.

PERSONNEL

The plant was designed and constructed under the direction of W. L. McFaul, City Engineer and Manager of the Waterworks of Hamilton. The principal assistants employed were W. W. VanEvery, mechanical and electrical engineer; H. S. Philips, designing engineer; and J. Stodart, waterworks engineer. The plant has been operated in conjunction with the High Lift Pumping Station by Jas. Bain, superintendent, and D. H. Matheson, chemist and bacteriologist, under the direction of the writer.

The general contractor was W. H. Yates Construction Co., Ltd., Hamilton, and other contractors as follows: filtration equipment, Francis Hankin & Co., Ltd., Montreal; penstocks and Venturi meter, W. J. Westaway Co., Hamilton; special castings and cast iron pipe, Gartshore-Thomson Pipe & Foundry Co., Ltd., Hamilton, and Canada Iron Foundries Ltd.; electrical motors and equipment, Canadian Westinghouse Co., Ltd., Hamilton.

(*Presented before the Canadian Section meeting, March 27, 1935.*)

SAFEGUARDING PUBLIC WATER SUPPLIES FROM SOURCE TO CONSUMER

By Arthur E. Gorman

(Engineer of Water Purification, Bureau of Engineering, Department of Public Works, Chicago, Ill.)

There are two well known axioms especially pertinent to water works to which everyone engaged in this profession should give serious consideration. They are (1) "A State of Change is a State of Danger," (2) "Eternal Vigilance is the Price of Safety."

But why is this so? Because water is a commodity on which both humans and animals depend for their daily existence. Because our present mode of living—in urban communities, at least, is predicated largely on the proper development of a public water supply. Because we are living in an era of rapid economic changes and because the immediate future holds for us prospects of tides in our environmental life not previously considered attainable for many years to come.

In previous presentations before groups of water works men the writer has emphasized the importance of (1) watching the weather in safeguarding water supplies, and also the necessity of being prepared for emergencies (2), in order to assure a safe supply of water to the ultimate consumer.

In the past the water works profession has bent its greatest efforts towards obtaining water from abundant sources of supply, and in so treating this water as to make it acceptable and safe for public and private uses. In meeting these requirements vast sums of public moneys have been invested in water works plants and structures, and properly so.

Most water works systems and equipments are built to meet conditions of strain in excess of those which might be expected in average operation. Units so designed usually will function satisfactorily with an average degree of intelligent operation. But still we continue to have water borne outbreaks of disease in cities where public investment in the water utility is high. What is the reason? If one

seeks, he will find, all too frequently, that the same standards of supervision over the safety of the water supply which are enforced in regard to protecting the source or in purifying the product were not carried through until the water reached the tap or outlet to the ultimate consumer. It is the water delivered through the consumer's tap or service which pays for this utility, and it is this water which should meet the strictest standards as to service and quality if the utility is to function as it should.

If one examines the investment in water works he will find that a large percentage is in the system for distributing the water. This is a hidden asset to be sure. But it is an asset of tremendous value and one deserving of more attention both from the standpoint of public health and property maintenance.

In the previous papers referred to, the writer has given special emphasis to meeting emergency conditions which relate to the source of supply and the purification of water. Therefore, he will touch only lightly on these matters and confine this paper largely to safeguarding the water in its final routing to the ultimate consumer.

A community which has invested in an adequate public water system has every right to expect that the officials in charge are taking the necessary precautionary measures to protect this investment in the best interest of the people. One takes it for granted that a competent water works superintendent has familiarized himself with the various potential operating risks which pertain to the source of his water supply, both as to quantity and quality. The literature is replete with references to emergency conditions which have been met and dealt with in this connection. The same is true as regards the operation of water purification plants.

Still we not infrequently learn of emergency conditions involving a shortage of water, interruption in service, or illness, resulting from failure during emergency periods to meet conditions of strain which a keen water works, superintendent, in the light of experience of others, might well have anticipated. For example, if a public water supply depends on a deep well, it is obviously the duty of the superintendent of that system to keep himself posted on other wells in the vicinity and especially to see that those which have been abandoned have been properly blocked and capped, to prevent pollution of the water bearing strata from which his water supply is obtained. Likewise, at a water purification plant, it is the duty of a good operator to familiarize himself with the character of run-off on the water shed serving

as the source of supply in order that the purification processes used may be adjusted to changes in the character and quality of the raw water after severe rains. With the general knowledge of the effectiveness of chlorination, especially for use during periods of emergency, the public has every right to assume that its water supply is protected by duplication of equipment and duplication of the water supply to that equipment if its successful operation depends on proper water service.

Several years ago the writer collaborated in the collection of information in regard to water borne outbreaks (3) in the United States and Canada during the decade 1920–1929 inclusive. To our surprise we learned that nearly 45.0 percent of those outbreaks in the United States were due, not to faulty plant operation, but actually to contamination of the water enroute to the ultimate consumer. Think of it!! clean, safe water polluted in the final distribution system where one would naturally assume there should be no hazard to health.

Supplying water to our citizens may be likened to supplying food to soldiers in time of warfare. For an army to operate successfully, each soldier must be fed regularly and the food he receives must be palatable, nourishing and safe. If it is otherwise the army will fail in its function as a fighting unit. The food as purchased by the quartermaster may be of the highest quality, but, if in the various channels of transmission to the soldier its quality becomes deteriorated through neglect or exposure to decay and pollution, the fighting capacity of the army will be promptly and seriously affected. Similarly, the health and well being of a community depends on the character and quality of the water supply in the individual homes, industries and commercial establishments. There must be no deterioration or contamination of the water supply of a community in distribution to the ultimate consumer. This is one of the great problems which confronts the water works and public health professions today, especially in the larger cities or in communities where large industrial establishments are served by the public water system.

HAZARDS IN DISTRIBUTION

This brings us back to the old story of a dual water supply systems cross connections, and allied hazards to the public health. They are the big bad wolves in the life of every water works. In spite of all the good work which has been done in the past, evidence is accumulating to indicate that we are approaching a period when their mute

and ominous existence may result in a new series of water borne outbreaks.

For over six years this country has been in the throes of a very serious economic depression. This has affected practically everyone, and especially industry and public utilities. There has been retrenchment all along the line; abandonment of entire plants or manufacturing units; weakening of supervisory control measures. In many cases maintenance work has been reduced to a point bordering on absurdity. During this period of low production there has been a decline in water usage and to some extent a removal of risks to water systems on industrial premises. However, industry has tried during this lull in productive forces to maintain its organization by diverting the efforts of its keenest men to research and developmental work. As a result there are clear indications that new products and new methods of manufacture will appear when the nation recovers economically. Since water is a basic utility serving industry, it is obvious that water works will be affected by this change. It is the duty and obligation of the water works superintendent to keep abreast of developments along these lines, as other utilities are doing. For example, many plants have been abandoned for years. These properties may be leased by new companies and operated on an entirely different basis from that which previously existed. Their water supply demands and uses will be changed; new hazards pertaining to water service may be created. If the water works superintendent is not alert and aware of this, serious contamination of the water system may result.

Plants which formerly existed may be entirely wrecked and water service connections abandoned, with resulting hazards to the water system. The writer recalls clearly the case of a brewery abandoned the year following the enforcement of the National Prohibition Act, when as the result of the dismantling of piping facilities on the premises a very serious potential hazard to the city water system was created. The conditions were such that it seemed as if the hand of Providence must have reached out to prevent an epidemic of severe proportions. It is the duty of the water works superintendent at this time, more than ever, to know for what purpose his water supply is used on every premise and the potential hazards which this use may cause in the public system.

The development of new equipment and processes is certain to have a definite bearing upon the use of water in the future. In particular

we have the advances in air conditioning for buildings, homes and theatres. Usually this calls for additional water for condensing purposes. If this water is obtained from the city water system we shall have the problem of additional consumption to be met, compensated for, of course, by increased revenue but complicated by the problem of re-using water discharged from the condensers under conditions which may jeopardize the public health.

In Chicago the use of water for condensing purposes has precipitated several new problems. Among them is the risk in the re-use of the condenser water for general domestic purposes within buildings as an economy item. Not infrequently the conditions under which the water is stored expose it to contamination. It was the re-use of condenser water with subsequent pollution through a cross connection which was a major factor in the famous amebic dysentery outbreak in 1933. There is the possibility that in the future a new refrigerant may be developed which will be a hazard to public health should it leak into the condensing water.

Everyone recognizes the fact that a water works system is not designed entirely to supply water for domestic purposes, but must be balanced to meet the other great function of water service—industrial uses and the fighting of fires. If the use of water for fire fighting were not given consideration our investments in water works plants and equipment would be very much smaller than they are at the present time. Therefore, in considering various potential health hazards to a public water supply system one must be reasonably tolerant, and seek a proper balance between public health, public safety and public welfare.

One of the best influences in promoting proper design in water works are the requirements of the various Board of Fire Underwriters. In most instances, from the standpoint of inspection of water usage on properties their work is well in advance of any ordinary water works organization. Their prime object is to prevent fires and in doing so water must be available in quantity and under special pressure conditions. Not infrequently in designing for fire protection purposes hazards have been created on certain premises which seriously jeopardize the public health.

No one, least of all the fire underwriters, will deny that, within reasonable limits, measures sponsored by health officials for safeguarding a public water supply against contamination, should take precedence over those deemed desirable by insurance officials for fire

fighting purposes. Our present difficulties in this connection seem to the writer to lie largely in the fact that the water works and public health officials have not always been as alert to their responsibilities in preventing epidemics as the insurance officials have been in providing adequate fire protection to meet a possible conflagration. True, the latter group operating on private funds of almost unlimited resources have more to work with in making premises inspections than the public water works or health official. But in the end the funds come from the ultimate consumer of water and the basic interests are essentially the same.

A water distribution system contains a myriad of service outlets which are theoretically supposed to deliver water under pressure to the consumer's premises. As long as this water flows in one direction, namely, from the distribution system to the consumer premise, there is little risk of contamination to the public water supply. However, when this flow becomes reversed a real hazard may exist. It is one which too frequently is not appreciated; one which requires alertness on the part of the water system superintendent to detect. In every large building where the direct pressure from the city mains is insufficient to give water service on the upper floors, some type of booster system exists. Under these conditions the elevation of this water is sufficient to cause a head greater than that existing in the city system. The possibility of leakage from the building to the city system through a check valve is ever present. If this water has been pumped and stored without exposing it to pollution usually no serious damage will result. But if the water is used for some purpose and then exposed to contamination in re-pumping and storage there is a definite danger that the public distribution system may be contaminated. Conditions of this kind are numerous in all large cities.

DUAL WATER SYSTEMS

There is the common and extremely dangerous case where on a property two water systems exist, one from the city system and the other from a polluted secondary source. Usually the latter water is used for fire protection or industrial purposes, with pressures much greater than those under which the city water is delivered. Here a "redhot" hazard would exist should the two systems become cross connected. The fact that they are not interconnected does not mean that they never will be. The probability of their being cross

connected either deliberately or innocently and the damaging consequences are sufficient to warrant strong regulations for the protection of the public water supply on such premises.

One precaution which might be adopted in plants where dual water systems exist to prevent misunderstandings as to piping would be to paint each system a distinctive color. It would not be necessary to paint the entire pipe, but only each fitting and at these points where pipe passes through a wall, floor, partition or roof. Obviously, while this means of identification is being followed out the system must be positively identified or serious cross connections might result. But when once properly done it is less probable that changes in plant operating personnel would be followed by confusion and misunderstanding as to the piping systems, which is so common in industrial establishments today.

FIVE RECENT OUTBREAKS OF WATER BORNE ILLNESS

In the last two years Chicago has experienced five outbreaks of water borne illness, resulting on a basis of conservative figures—in approximately 75 deaths, illness to about 1665 persons and causing several thousand to become infested with cysts of E. histolytica. These outbreaks occurred on premises where dual water systems existed and in four of the five outbreaks were due to cross connections. In two of the outbreaks the cross connections were made with the knowledge that a polluted water and the domestic water systems were being interconnected. In the third, it is reported that the cross connection had been in existence for over a quarter century apparently without knowledge of its extreme hazard; in the fourth outbreak of recent occurrence a plant which had previously been shut down was started up again by persons not fully familiar with the piping conditions. The result was that under the new operating conditions a cross connection formerly involving a remote risk became operative in a positive way.

This unenviable record in such a short period of time indicates the hazard of supplying any premises with water directly from public mains when a secondary or polluted source of supply also exists on that premise. It is the writer's opinion that on all premises of this type the service from the city mains should be through a receiving reservoir properly protected against possible contamination. Regulations of this character are now being considered in Chicago. As proposed, this receiving reservoir would consist of a covered and

properly vented tank, with water service from the city mains received through an automatically operated float valve to maintain an established elevation of water in the tank. The overflow and vent pipe would be installed with no direct connection to a sewer, and be properly screened to prevent entrance of rodents and insects. The inlet line would be so installed as to prevent back siphonage of water into the city mains.

The installation of such a tank would of course mean the loss of city pressure on the premises, but this loss is one which would have to be met as an operating charge against the property for maintaining a hazardous secondary water system. No bypass should be permitted around the tank. Such a tank would be installed for all water service on a premise where a secondary water supply is maintained, or where the city supply is re-used and stored at a pressure greater than that in the public water system. While it is realized that such a regulation has the disadvantage of requiring extra space, increased operating costs and in some cases larger or additional service connections, it is considered reasonable in the interests of public health. No serious objection to it is anticipated.

SPECIAL HAZARDS

There is another type of hazard which exists in Chicago and also in other large cities, namely outside hydrants on buildings with fittings for fire boats to pump into at times of fire. These outside connections are commonly known as "Siamese Twin" connections. They connect to the fire protection system in the building which system is also directly connected to the city mains.

In the writer's opinion they constitute a serious potential health hazard, because if used the pressure from the fire boat would be several times that in the city mains, with a single check valve as the only protection against the flow of grossly polluted river water into the city system. Ordinarily, they are used in Chicago very little and it is expected that no difficulty will be experienced in obtaining their early elimination. One must distinguish between outside hydrants into which a fire boat may pump and the customary outside hydrants on buildings in which the fire department engines may pump water from the city mains to supplement the fire protection system within a building. The latter type of connection has real fire prevention possibilities with remote health risks and under normal conditions no serious hazard exists in their use.

CONCLUSIONS

In conclusion, it is desired to emphasize to water works men that the advent of economic recovery may bring with it problems involving new hazards of pollution to their water system which deserve the keenest consideration. It is most important that water works superintendents know the uses to which water is put on all consumer premises, especially at industrial plants; also the possible hazards of these uses to the public water supply system.

To accomplish this purpose it is suggested that he discuss water service and pollution problems frankly with plant superintendents, representatives of the fire underwriters and state department of health engineers. By taking an aggressive, yet friendly attitude in this matter, a water superintendent has much to gain and may prevent a serious water borne epidemic.

On properties where a polluted water supply exists all service connections from a public distribution system should be protected so that under no circumstances could the water used on the consumer's premises return to the city system. A receiving reservoir properly safeguarded against local contamination and designed to meet water service requirements is recommended as a means of protecting the public system.

The public health hazards of fire boat connections to outside hydrants on large buildings is deserving the early attention of water works, public health, designing and fire prevention engineers.

(*Presented before the Wisconsin Section meeting, November 6, 1935.*)

REFERENCES

(1) Watching the Weather in Safeguarding Public Water Supply. Jour. New England W. W. Assoc., Jan., 1930.

(2) Meeting Emergencies in Water Works, Jour. Amer. W. W. Assoc., February, 1934.

(3) Significance of Waterborne Typhoid Fever Outbreaks, Wolman and Gorman, Jour. Amer. W. W. Assoc., February, 1931.

PREPARATION AND CONDITIONING OF ALUMINUM SULPHATE SOLUTION BEFORE APPLYING TO WATER

By Carl Leipold

(Superintendent, Water Filtration Plant, Winnetka, Ill.)

Preparation of alum solution, as a preliminary stage in the process of coagulation, is an important feature of the modern water purification plant, since the clarification of the water is dependent upon the dissolving of the alum, proper mixing of the solution and uniformity of its application. For preparation of alum solution, either dry feed or solution feed equipment is used.

In making laboratory and plant studies of alum floc formation with mechanical agitation,[1] it was noted that floc results varied with the method of preparation of the alum solution. To determine the most effective method of preparing alum solution, the writer has made laboratory experiments and plant studies with solution feed and dry feed equipment at the Winnetka, Illinois, water filtration plant.

SOLUTION FEED

A 3 per cent alum solution is prepared in two concrete tanks of 800 gallons capacity each, having a dissolving box located at the top of the tank, a gravel filter connected to the tank outlet pipe line, a constant level tank and a manual control orifice (figure 1). An air blower operating at 850 R.P.M., having a capacity of 45 to 58 cubic feet displacement per minute at 1 to 3 pounds pressure, is used for mixing the solution.

Two types of dissolving boxes were used for dissolving the alum. In dissolving box No. 1 (figure 2) a manifold of 1-inch brass pipe drilled with $\frac{3}{16}$-inch holes was assembled at the bottom of the box. For dissolving the alum, agitation was provided for by the water flowing upward through the spray pipe openings. The solution overflowed from the box and passed into the solution tank through a copper screened baffle.

[1] Jour. A. W. W. A., vol. 26, no. 8, page 1070.

In dissolving box No. 2 (figure 3) air agitation was provided for by a manifold of $\frac{1}{2}$-inch galvanized pipe drilled with $\frac{1}{8}$-inch holes.

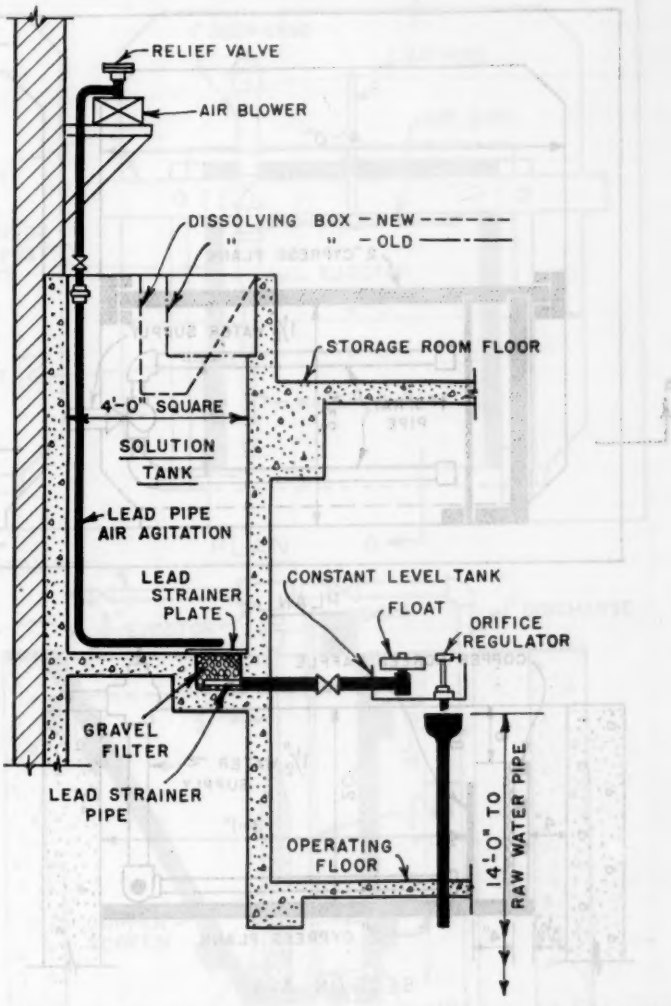


FIG. 1. CROSS SECTION OF ALUM SOLUTION FEED EQUIPMENT AT WINNETKA

The air grid was located at the bottom of the box and was connected to the solution tank air blower. The same $\frac{3}{4}$-inch galvanized pipe used for filling the box with water, was used for removing the solution

from the box into the solution tank with a $\frac{3}{4}$-inch water ejector connection.

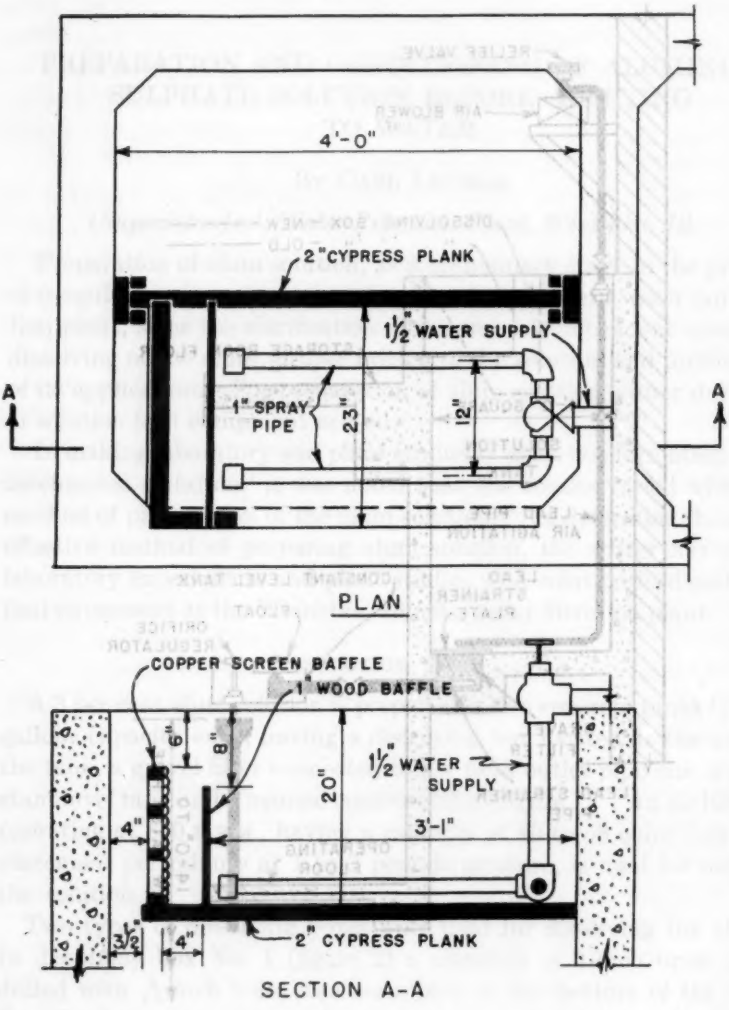


FIG. 2. SOLUTION FEED TANK; DISSOLVING BOX NO. 1

SOLUTION STRENGTH

To determine whether the methods of dissolving the alum and air agitation of the prepared solution produced a uniform strength of

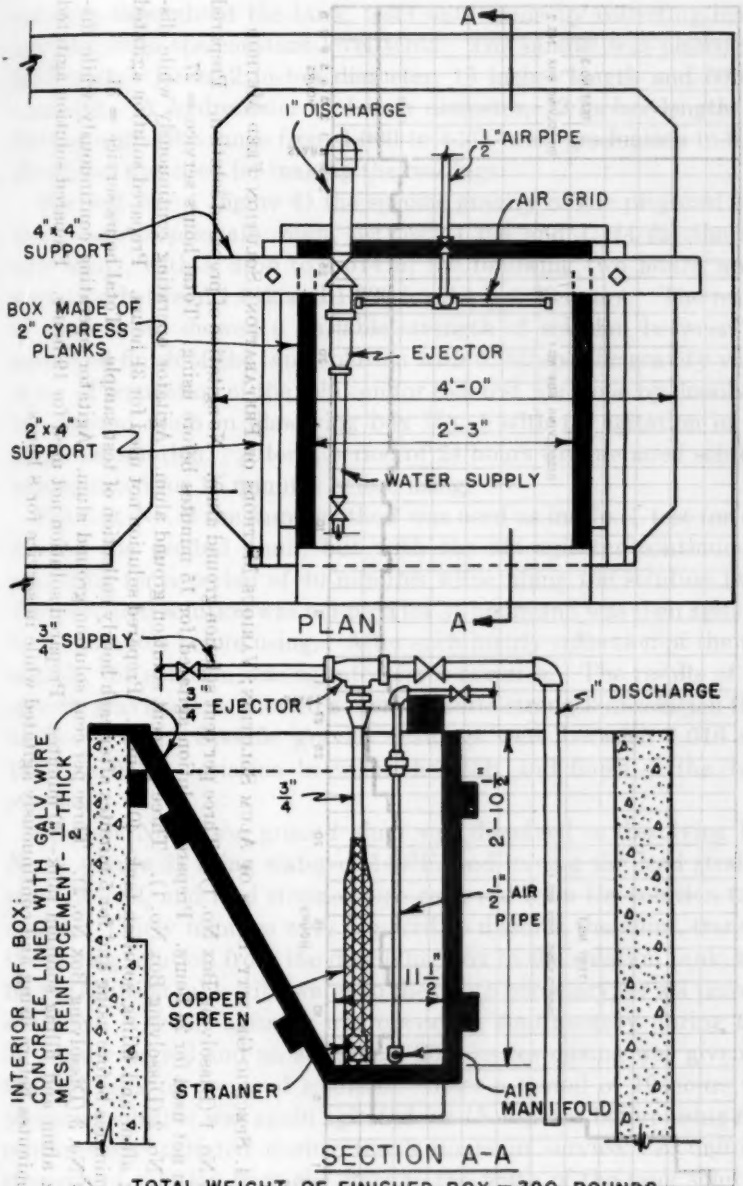


FIG. 3. SOLUTION FEED TANK; DISSOLVING BOX NO. 2

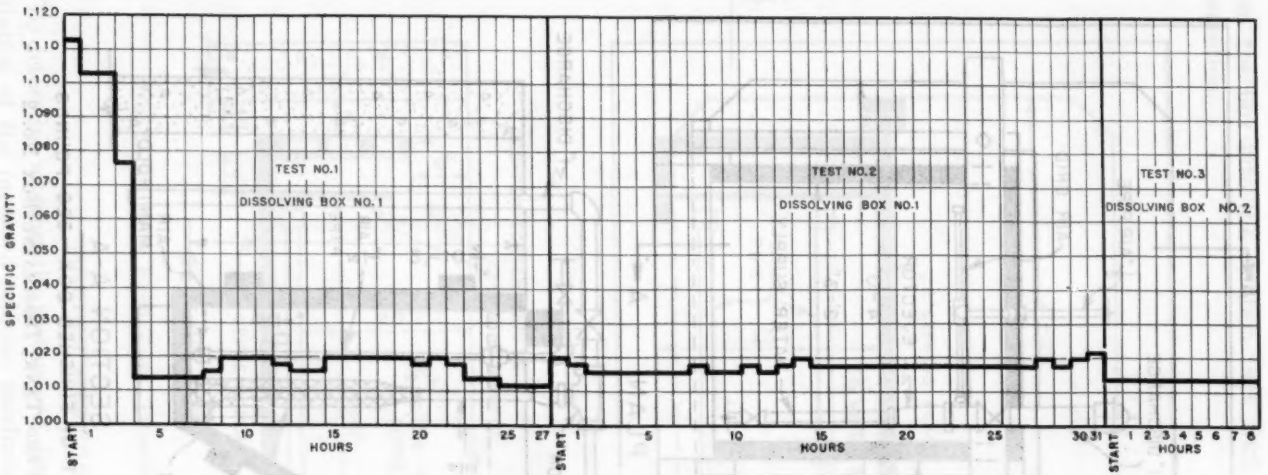


FIG. 4. SPECIFIC GRAVITY TESTS OF ALUM SOLUTION; VARIOUS METHODS OF PREPARATION; SOLUTION FEED METHOD

Test No. 1 (Dissolving Box No. 1). Three per cent solution ground alum. No agitation of prepared solution. Prepared solution not used for 24 hours. Prepared solution agitated for 15 minutes before using. Total hours service = 27.

Test No. 2 (Dissolving Box No. 1). Three per cent solution ground alum. Agitator operating continuously while dissolving alum and filling solution tank—40 minutes. Prepared solution not used for 31 hours. Prepared solution agitated for 15 minutes before using and for 5 minutes after each hourly collection of test sample. Total hours service = 31.

Test No. 3 (Dissolving Box No. 2). Three per cent solution ground alum. Agitator operating continuously while dissolving alum and filling solution tank—30 minutes. Prepared solution not used for 19 hours. Prepared solution agitated for 15 minutes before using and continuously agitated while in service for 8 hours.

solution throughout the tank, tests were made by collecting hourly samples from the constant level tank. The sample was placed in a hydrometer jar of 2 inches diameter, 15 inches length and 600 cc. capacity. A hydrometer of 1-inch diameter, 12 inches length and having a gravity range from 1.000 to 1.200 with graduation in 0.002 divisions, was used for making the readings.

For test No. 1 (figure 4) the specific gravity of the prepared solution was exceptionally high, between 1.104 and 1.114 for the first two hours, with a drop to 1.014 in the following two hours, and a variation between 1.012 and 1.020 for the last 23 hours. The results of these tests showed a variable strength of solution between the start and finish of the tank solution with 0.102 specific gravity variation. Preparation of the solution for this test was done by dissolving the ground alum in dissolving box No. 1 with no agitation of the prepared solution. After a period of 24 hours the prepared solution was agitated for 15 minutes before using.

For test No. 2 the same method was used as in No. 1 test for dissolving the ground alum, but with the air agitator continuously operating for a period of 40 minutes while filling the solution tank. The prepared solution was not used for 31 hours and was then agitated for 15 minutes before using. After each hourly collection of the test sample, the solution was agitated for 5 minutes. The results of the specific gravity tests showed a more uniform strength of solution than in No. 1 test. Specific gravity readings were between 1.016 and 1.022, a 0.006 variation between the start and finish of the tank solution.

For test No. 3 the ground alum was dissolved in dissolving box No. 2 (figure 3) using water of 140°F. and having the lead strainer plate, gravel, and lead strainer pipe removed from the solution tank outlet. Thirty minutes were required to dissolve the alum, transfer the strong solution from the dissolving box to the solution tank, and fill the solution tank with water to the depth necessary for a 3 percent solution. The air agitator was operating continuously during this 30 minute period and an additional 30 minutes mixing was given to the full tank of prepared solution. After a period of 19 hours the prepared solution was again agitated for 15 minutes before using and continuously agitated during the 8 hours in service. A uniform strength of solution from the start to the finish of the tank solution was obtained by this method of preparation.

The results of the specific gravity tests for the solution feed method,

indicate that agitation is an important feature both in the dissolving box and solution tank for preparation of alum solution. Agitation in the dissolving box causes rapid dissolving action. Continuous agitation in the solution tank at the time of filling and during the service period, is required to obtain a uniform strength of solution throughout the tank. The dissolving box, when of tight construction, retains the alum and assures complete dissolving of the alum before discharging the solution into the solution tank.

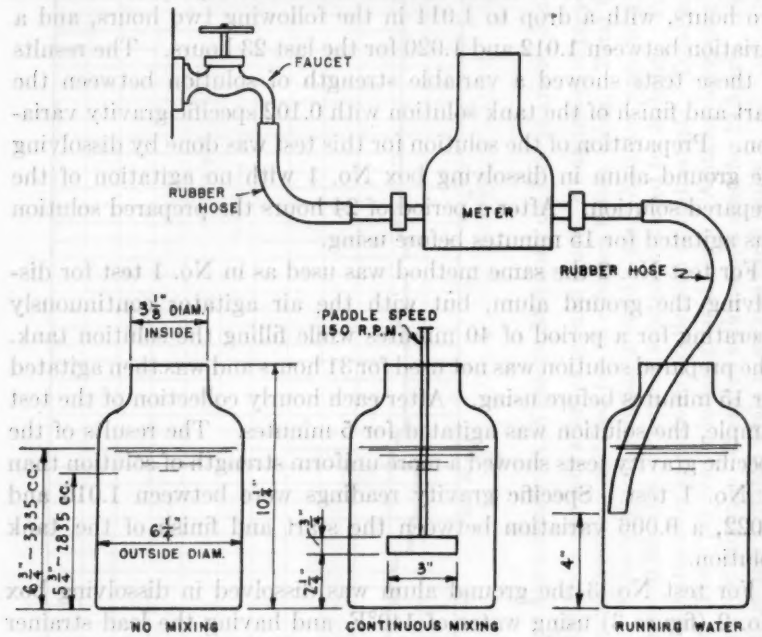


FIG. 5

It is important to clean the solution tank, the constant level tank, and the tank orifice, after each service period to assure uniformity of solution application. In making the specific gravity tests it was shown that the hydrometer should not be cleaned nor handled other than at the tip of the graduated tube, during the period of testing the tank solution.

ALUM DISSOLVING TESTS

Three methods were used, namely, (1)—no mixing, (2)—continuous mixing, and (3)—running water, to determine the most efficient

TABLE 1

Alum dissolving test

Ground alum. Weight 100 grams

No mixing. Filtered water 3335 cc. Solution strength 3 percent

Filtered water, pH	7.7	7.6	7.7	7.7	7.7	7.7
Temperature, °F:						
Start	35	60	80	100	120	142
Finish	41	64	78	82	112	118
Dissolving period, hours	24 hours and not dissolved	24 hours and not dissolved	24 hours and not dissolved	24 hours and not dissolved	22	20

Continuous mixing. Filtered water 3335 cc. Solution strength 3 percent

Filtered water, pH	7.6	7.6	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	36	60	80	100	120	132
Finish	42	60	80	100	120	132
Dissolving period, hours	.500 = 30 minutes	.166 = 10 minutes	.116 = 7 minutes	.050 = 3 minutes	.041 = 2½ minutes	.029 = 1¾ minutes

Running water

Filtered water, pH	7.6	7.5	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	47	62	80	100	120	145
Finish	50	60	80	100	120	140
Water used:						
Total gallons	120	119	60	45	34	24
G. P. M	1.2	1.2	1.3	1.5	1.7	1.6
Dissolving period, hours	1.66	1.66	.750 = 45 minutes	.500 = 30 minutes	.333 = 20 minutes	.250 = 15 minutes

Lump alum—two pieces. Weight 100 grams (50 grams each)

No mixing. Filtered water 3335 cc. Solution strength 3 percent

Filtered water, pH	7.7	7.7	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	35	60	80	100	120	140
Finish	44	61	78	100	110	119
Dissolving period, hours	10.0	7.0	5.50	5.0	2.50	2.50

Continuous mixing. Filtered water 3335 cc. Solution strength 3 percent

Filtered water, pH	7.7	7.7	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	35	60	80	100	120	140
Finish	50	66	78	94	110	125
Dissolving period, hours	2.16	1.75	1.25	.666 = 40 minutes	.583 = 35 minutes	.50 = 30 minutes

Running water

Filtered water, pH	7.6	7.6	7.6	7.7	7.7	7.7
Temperature, °F.:						
Start	47	60	80	100	120	138
Finish	52	60	80	100	120	136
Water used:						
Total gallons	76	192	120	78	56	45
G. P. M	0.6	1.6	1.5	1.3	1.4	1.8
Dissolving period, hours	2.16	2.0	1.33	1.0	.666 = 40 minutes	.416 = 25 minutes

TABLE 1—*Continued*

Lump alum—one piece. Weight 170 grams

No mixing. Filtered water 2835 cc. Solution strength 6 percent

Filtered water, pH	7.7	7.7	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	35	60	80	100	120	134
Finish	44	61	72	94	116	122
Dissolving period, hours	11.0	8.50	8.0	3.75	3.50	3.75

Continuous mixing. Filtered water 2835 cc. Solution strength 6 percent

Filtered water, pH	7.7	7.7	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	39	60	80	100	120	140
Finish	52	64	77	92	114	119
Dissolving period, hours	2.0	1.75	1.50	.91 = 55 minutes	.75 = 45 minutes	.66 = 40 minutes

Running water

Filtered water, pH	7.6	7.6	7.6	7.7	7.7	7.7
Temperature, °F.:						
Start	49	60	80	100	120	136
Finish	50	60	80	100	120	142
Water used:						
Total gallons	217	232	154	87	62	42
G. P. M.	1.0	1.4	1.4	1.4	1.2	1.0
Dissolving period, hours	3.50	2.75	1.83	1.0	.83 = 50 minutes	.66 = 40 minutes

Lump alum—four pieces. Weight 100 grams (25 grams each)

No mixing. Filtered water 3335 cc. Solution strength 3 percent

Filtered water, pH	7.7	7.7	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	35	60	80	100	120	140
Finish	44	61	74	99	114	116
Dissolving period, hours	12.0	12.0	11.0	10.0	7.0	5.0

Continuous mixing. Filtered water 3335 cc. Solution strength 3 percent

Filtered water, pH	7.7	7.5	7.6	7.7	7.7	7.7
Temperature, °F.:						
Start	35	60	80	100	120	140
Finish	50	64	80	96	112	132
Dissolving period, hours	1.91	1.0	.750 = 45 minutes	.666 = 40 minutes	.416 = 25 minutes	.250 = 15 minutes

Running water

Filtered water, pH	7.6	7.6	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	52	60	80	100	120	134
Finish	48	60	80	100	120	135
Water used:						
Total gallons	134	126	75	45	35	27
G. P. M.	1.1	1.4	1.1	1.1	1.1	1.1
Dissolving period, hours	2.0	1.50	1.08	.66 = 40 minutes	.50 = 30 minutes	.41 = 25 minutes

TABLE 1—*Concluded*

Lump alum—one piece. Weight 100 grams

No mixing. Filtered water 3335 cc. Solution strength 3 percent						
Filtered water, pH	7.7	7.7	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	35	60	80	100	120	140
Finish	44	61	76	94	108	120
Dissolving period, hours	10.0	6.75	5.75	3.0	3.0	2.16
Continuous mixing. Filtered water 3335 cc. Solution strength 3 percent						
Filtered water, pH	7.7	7.7	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	35	60	80	100	120	140
Finish	48	66	77	93	108	126
Dissolving period, hours	2.58	1.91	1.50	.75 = 45 minutes	.666 = 40 minutes	.50 = 30 minutes
Running water						
Filtered water, pH	7.6	7.6	7.7	7.7	7.7	7.7
Temperature, °F.:						
Start	42	60	80	100	120	148
Finish	43	60	80	100	120	135
Water used:						
Total gallons	209	183	102	66	52	41
G. P. M.	1.2	1.2	1.0	1.3	1.0	1.0
Dissolving period, hours	2.91	2.50	1.66	.83 = 50 minutes	.83 = 50 minutes	.66 = 40 minutes

method of dissolving alum (figure 5). Various sized alum and various water temperatures were also used for these tests.

One gallon capacity bottles, round shape, of clear flint glass with wide mouth, metal screw cap type, were used for the containers.

The lump alum used for these tests was not of uniform shape and of the same solid qualities. Some pieces had cracks and irregular edges. The main object was to have pieces of alum of the required test weight and which were representative samples from the 200 pound bags of alum. By chipping and breaking with a hammer, the 144 pieces of lump alum used for these tests varied in size, and weight from 25 to 170 grams each.

In No. 1 method—*no mixing*, the bottles containing the water of test temperature and the weighed alum, were placed in the laboratory, or in the filter pipe gallery, or in the incubator, for temperature control. No disturbance of the water or alum in the bottle, occurred during the dissolving period.

In No. 2 method—*continuous mixing*, the water of test temperature and the weighed alum in the bottle, were agitated continuously until the alum was dissolved. The laboratory mixer paddle size of $\frac{3}{4}$-inch

by 3 inches equals 7 cubic feet bottle capacity per square foot paddle surface area with 3335 cc. water, and 6 cubic feet bottle capacity per square foot paddle surface area with 2835 cc. water. The paddle length was 50 percent of the bottle diameter and the paddle speed at 150 R.P.M. equals 117 F.P.M. at outer ends of paddle.

In No. 3 method—*running water*, the water was taken from the laboratory faucet which permitted the use of hot, cold or tempered water; and for water flow control, a rubber hose connection was made from the faucet swinging spout to the water meter inlet and from the water meter outlet to the inside of the bottle, the end of the hose being 4 inches from the bottle bottom. The bottle was placed in the laboratory sink to permit drainage of the overflow, and the water meter had a special disc for hot water tests.

The results of No. 1 method—*no mixing*, showed that a longer period was required for dissolving the smaller sized alum regardless of the water temperature. The tests with water having 35° to 44°F. temperature, alum weight of 100 grams, and 3335 cc. water equals 3 percent solution, showed the following dissolving periods:

	hours
Ground alum	24*
Lump alum (four 25-gram pieces)	12
Lump alum (two 50-gram pieces)	10
Lump alum (one 100-gram pieces)	10

* Alum not dissolved.

The results of No. 2 method—*continuous mixing*, showed opposite results than No. 1 method. A longer dissolving period was required for the larger sized alum regardless of the water temperature. The tests with water having 35° to 50°F. temperature, alum weight of 100 grams, and 3335 cc. water equals 3 percent solution, showed the following dissolving periods:

	minutes
Ground alum	30
Lump alum (four 25-gram pieces)	115
Lump alum (two 50-gram pieces)	130
Lump alum (one 100-gram pieces)	155

The results of No. 3 method—*running water*, showed results similar to No. 2 method. A longer dissolving period was required for the larger sized alum regardless of water temperature. The tests with water having 42° to 52°F. temperature, alum weight of 100 grams,

and water flow of 1.2 gallons per minute, showed the following dissolving periods:

	minutes
Ground Alum	100
Lump alum (four 25-gram pieces)	120
Lump alum (two 50-gram pieces)	130
Lump alum (one 100-gram pieces)	175

The three methods of dissolving alum, showed that the *continuous mixing* and the *running water* methods perform similar results for dissolving *lump alum*, but *continuous mixing* is the most effective method of the three for dissolving *ground alum*.

These tests also showed that the water temperature is a controlling factor for the alum dissolving period. A reduction of 58 to 78 percent of time or 7 to 7.84 hours is effected to dissolve alum of various size with an increase in water temperature from 35° to 140°F. using No. 1 method; a reduction of 72 to 94 percent of time or 28 to 125 minutes using No. 2 method; and a reduction of 77 to 85 percent of time or 85 to 135 minutes using No. 3 method.

PADDLE POSITIONS AND SPEEDS FOR DISSOLVING ALUM

Laboratory tests were made to determine the effect of the paddle placed at various positions relative to the water depth using mechanical agitation for dissolving ground alum to a 3 percent solution (figure 6). Various paddle speeds also were used, namely, 70 R.P.M. equals 55 F.P.M. at outer ends of paddle, 140 R.P.M. equals 110 F.P.M. at outer ends of paddle, and 280 R.P.M. equals 220 F.P.M. at outer ends of paddle. The paddle size of $\frac{3}{4}$-inch by 3-inch equals 7 cubic feet bottle capacity per square foot paddle surface area, and paddle length equals 50 percent of the bottle diameter. The water temperature was 44°F. in all tests. The paddle was placed in three different positions, namely, No. A position was with paddle $\frac{1}{4}$-inch from the bottle bottom, No. B was 1-inch from the bottle bottom, and No. C was 2 inches from the bottle bottom.

With paddle speed at 70 R.P.M., the No. B paddle position test showed a 23 percent or 19 minutes shorter dissolving period than the No. A paddle position test, and a 21 percent or 17 minutes shorter dissolving period than the No. C paddle position test.

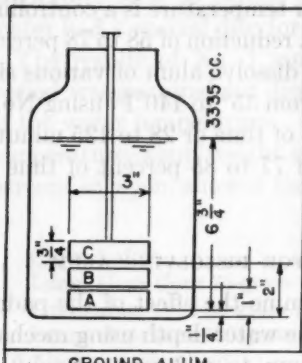
With paddle speed at 140 R.P.M., the No. B paddle position test showed an 80 percent or 36 minutes shorter dissolving period than the

No. A paddle position test, and a 62 percent or 15 minutes shorter dissolving period than the No. C paddle position test.

With paddle speed at 280 R.P.M., the dissolving periods were the same for all three paddle position tests.

At 70 R.P.M. paddle speed, the ground alum would settle and collect at the bottle bottom regardless of the paddle position.

At 140 R.P.M. paddle speed, the ground alum would settle and collect at the bottle bottom outer edge with paddle position No. A; would settle and collect at the bottle bottom center with paddle position No. B; would settle and collect at the bottle bottom center and outer edge with paddle position No. C.



WATER TEMPERATURE °F.		PADDLE POSITION			PADDLE SPEED R.P.M.			DISSOLV. PERIOD
START	FINISH	A	B	C	70	140	280	MINUTES
44	49	■			■			82
44	50	■				■		45
44	44	■					■	2½
44	49		■		■			63
44	44		■			■		9
44	44		■				■	2½
44	49			■	■			80
44	46			■		■		24
44	44			■			■	2½

GROUND ALUM WEIGHT—100 GRAMS

FILTERED WATER 3335 CC.—$_{p}$H = 7.6

FIG. 6. ALUM DISSOLVING TEST; CONTINUOUS MIXING; VARIOUS PADDLE POSITIONS AND PADDLE SPEEDS

At 280 R.P.M. paddle speed, the ground alum was kept in good suspension regardless of the paddle position.

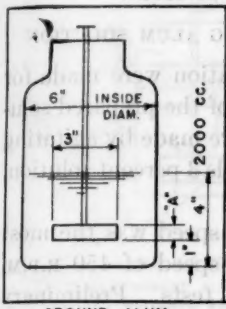
These tests indicating that high paddle speed is the most effective method for rapid dissolving of ground alum using mechanical agitation. For rapid dissolving action it is necessary to keep the ground alum in suspension and the particles away from one another. These tests also showed that paddle position and paddle speed are controlling factors for dissolving alum.

PADDLE SIZE AND SPEEDS FOR DISSOLVING ALUM

Laboratory tests were made to determine the effect of various size paddles and various paddle speeds for dissolving ground alum

(figure 7). All 8 paddles used for these tests had a 3-inch length, but the widths varied between $\frac{1}{32}$- and 1-inch. Water having a temperature of 54°F. was used for all tests with 60 grams ground alum and 2000 cc. water equals 3 percent solution.

PADDLE LENGTH = 50 PERCENT OF BOTTLE DIAMETER



DISSOLVING PERIOD MINUTES	PADDLE "A" INCHES	PADDLE SPEED — R.P.M. 70	TEMP. °F. FINISH	150	TEMP. °F. FINISH	250	TEMP. °F. FINISH	350	TEMP. °F. FINISH	450	TEMP. °F. FINISH	550	TEMP. °F. FINISH
	1/32					55	59	21	60	4.75	56	2.50	55
	1/16					26	58	4	56	2.50	56	1.75	55
105	1/8	62	33	61	6	56	2	56	1.75	55			
94	1/4	62	20	61	2.75	56	1.75	55	1.75	55			
78	3/8	62	11	61	2.25	56							
68	1/2	61	3.50	60	1.83	55							
58	3/4	60	3.66	58	1.75	55							
28	1	60	2.25	58	1.75	55							

GROUND ALUM 60 GRAMS — FILTERED WATER 2000 cc. — pH = 7.6

SOLUTION STRENGTH = 3%

WATER TEMPERATURE AT START = 54°F.

RISE OF WATER IN BOTTLE CAUSED BY MIXING ACTION—VARIOUS PADDLE SIZES AND PADDLE SPEEDS

PADDLE LENGTH = 50 PERCENT OF BOTTLE DIAMETER

PADDLE SIZE	PADDLE SPEED — R.P.M. 70 RISE—INCH	70 RISE—% OF WATER DEPTH	150 RISE—INCH	150 RISE—% OF WATER DEPTH	250 RISE—INCH	250 RISE—% OF WATER DEPTH	350 RISE—INCH	350 RISE—% OF WATER DEPTH	450 RISE—INCH	450 RISE—% OF WATER DEPTH	550 RISE—INCH	550 RISE—% OF WATER DEPTH
1/32 X 3					0	0	1/16	1.5	1/8	3.1	1/4	6.2
1/16 X 3					1/16	1.5	1/8	3.1	5/16	4.7	3/8	9.4
1/8 X 3	0	0	0	0	1/8	3.1	5/16	7.8	1/2	12.5		
1/4 X 3	0	0	1/16	1.5	3/16	4.7	1/2	12.5	5/8	15.6		
3/8 X 3	0	0	1/16	1.5	1/4	6.2						
1/2 X 3	0	0	1/8	3.1	5/16	7.8						
3/4 X 3	0	0	1/8	3.1	3/8	9.4						
1 X 3	0	0	3/16	4.7	1/2	12.5						

FIG. 7. ALUM DISSOLVING TEST; CONTINUOUS MIXING; VARIOUS PADDLE SIZES AND PADDLE SPEEDS

With paddle speed at 70 R.P.M., the alum was dissolved in 28 minutes using the 1-inch paddle, whereas with the $\frac{1}{8}$-inch paddle, 105 minutes was required or 77 minute longer dissolving period.

With paddle speed at 150 R.P.M., the alum was dissolved in $2\frac{1}{4}$ minutes using the 1-inch paddle, whereas 33 minutes were required with the $\frac{1}{8}$-inch paddle or 1366 percent longer dissolving period.

With paddle speed at 250 R.P.M. the alum was dissolved in $1\frac{3}{4}$

minutes using the 1-inch paddle, whereas a paddle speed above 550 R.P.M. or more than a 120 percent increase was required to dissolve the alum in the same time using the $\frac{1}{32}$-inch paddle.

These tests showed that paddle size and speed are controlling factors for dissolving ground alum.

DETENTION PERIODS OF MIXING FOR PREPARING ALUM SOLUTION

Various detention periods of mechanical agitation were made for preparing alum solution to determine the effect of the prepared solution on floc formation (figure 8). Four tests were made by agitating 60 grams of ground alum and 2000 cc. water equals 3 percent solution, for 5, 10, 20 and 30 minute periods.

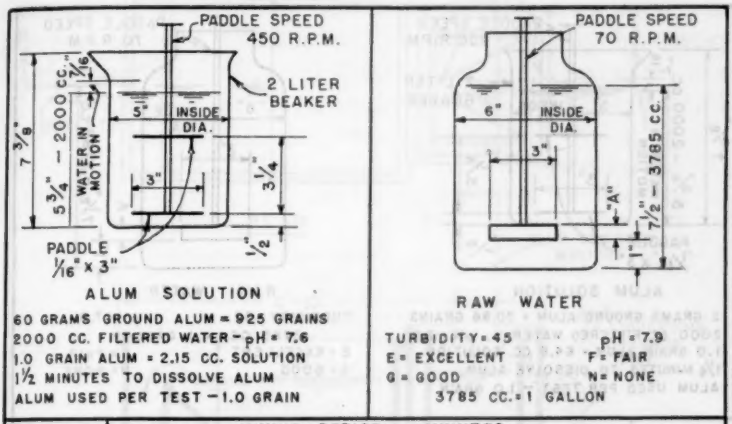
As previous tests showed that a high paddle speed was the most effective for dissolving ground alum, a paddle speed of 450 R.P.M. was used for these alum solution preparation tests. Preliminary tests with various size paddles operating at 450 R.P.M., showed that the agitation action by two smaller size paddles placed near the bottom and near the top of the water, caused a lower rise of the water in the beaker, than the one large paddle placed near the bottom of the water. Also the vortex was greatly eliminated with the two small sized paddles.

For preparation of the alum solution, the weighed ground alum placed in the 2 liter beaker containing the water, was agitated with the two $\frac{1}{16}$-inch by 3-inch paddles assembled $3\frac{1}{4}$ inches apart on the paddle shaft operating at 450 R.P.M. (figure 8). With this arrangement the alum was dissolved in $1\frac{3}{4}$ minutes.

For floc tests, 2.15 cc. alum solution, equal 1.0 grain alum, was taken from the beaker after the 5, 10, 20, and 30 minutes high speed mixing period and placed in the round shaped bottles containing one gallon raw water of 45 turbidity and 7.9 pH value. The paddles used in the bottle for the floc tests had a length of 3 inches for all tests and different widths of $\frac{1}{32}$-, $\frac{1}{8}$-, $\frac{3}{8}$- and $\frac{3}{4}$-inch.

The results of the tests for detention period of mixing required for alum floc formation with the various sized paddles operating at 0.91 F.P.S. peripheral speed, paddle length 50 percent of the bottle diameter, and using the 3 percent prepared alum solution after a 5 minute high speed mixing period are shown in figure 8. Alum floc tests using the 3 percent prepared alum solution after the 10, 20, and 30 minute high speed mixing periods, showed similar floc results as the tests using the prepared alum solution after a 5 minute high speed

mixing period. These tests indicated that prolonged high speed mixing of the alum solution in the preparation stage, has no preventive or retarding effect on the formation of alum floc in raw water using mechanical agitation.



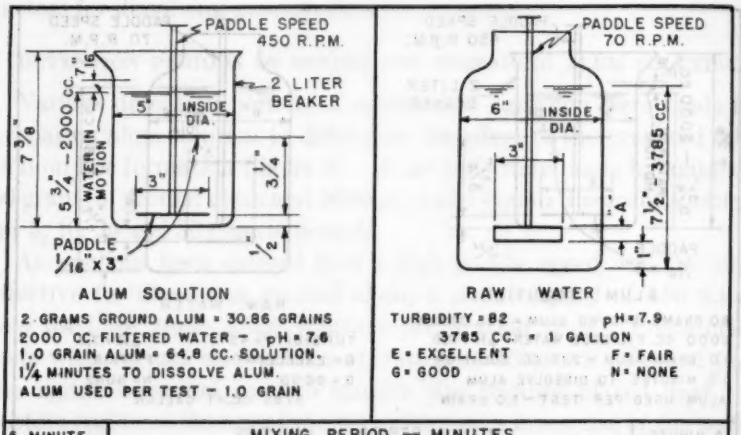
		MIXING PERIOD — MINUTES												
5 MINUTE ALUM SOLUT.		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	E	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								
10 MINUTE ALUM SOLUT.		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	E	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								
20 MINUTE ALUM SOLUT.		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	E	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								
30 MINUTE ALUM SOLUT.		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	E	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								

FIG. 8. ALUM FLOC TEST; VARIOUS MIXING PERIODS FOR DISSOLVING AND PREPARING ALUM BEFORE USING ALUM SOLUTION WITH RAW WATER; 3.0 PERCENT ALUM SOLUTION

SOLUTION STRENGTH

To determine whether the strength of the solution prepared by the high speed mechanical agitation method has any effect on the floc formation, laboratory tests were made with a 0.1 percent solution.

The same equipment and procedures were used as in the 3 percent solution tests. These tests showed similar results to the 3 percent solution tests for detention periods required for floc formation (figure 9), indicating that solution strength (below 3 percent) is not a con-



5 MINUTE ALUM SOLUT.		MIXING PERIOD — MINUTES												
		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	F	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								
10 MINUTE ALUM SOLUT.		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	F	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								
20 MINUTE ALUM SOLUT.		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	F	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								
30 MINUTE ALUM SOLUT.		2	4	6	8	10	12	14	16	18	20	22	24	26
FLOC	PADDLE "A" INCHES 1/32	N	N	N	N	N	N	F	F	F	G	G	G	E
	1/8	N	N	N	N	F	F	G	E					
	3/8	N	N	N	F	G	E							
	3/4	N	N	F	G	E								

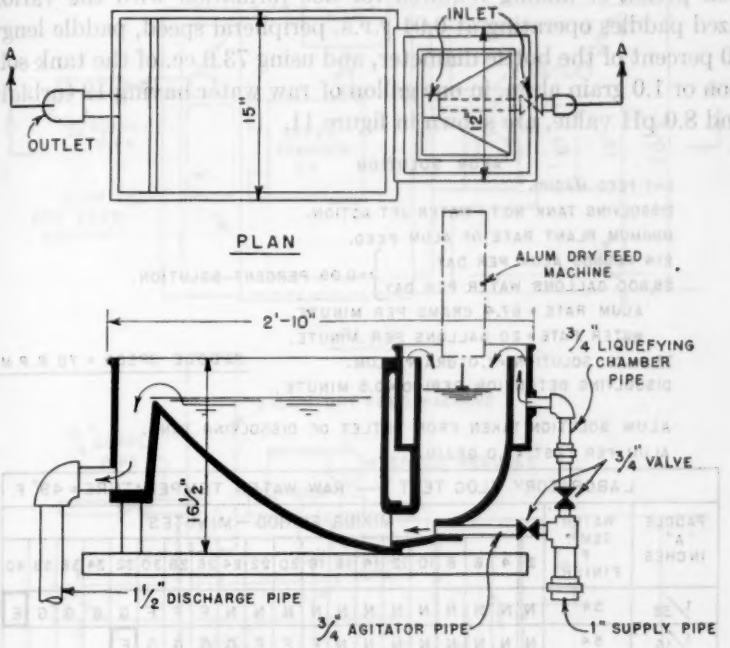
FIG. 9. ALUM FLOC TEST; VARIOUS MIXING PERIODS FOR DISSOLVING AND PREPARING ALUM BEFORE USING ALUM SOLUTION WITH RAW WATER; 0.1 PERCENT ALUM SOLUTION

trolling factor for alum floc formation with solution prepared by the high speed mechanical agitation method.

DRY FEED

The water plant dry feed machine used for the ground alum, has a storage bin of 8 cubic feet capacity and an alum feed control assembly

with a maximum rate of feed of 180 pounds per hour. The dissolving tank No. 1 (figure 10) has a capacity of 10 gallons with agitation by water jet action.



VALVE OPENING		TOTAL GALLONS WATER		DETENTION PERIOD DISSOLVING TANK MINUTES	WATER PRESSURE AT DRY FEED MACHINE POUNDS
3/4" LIQUEFYING CHAMBER VALVE	3/4" AGITATOR VALVE	24 HOURS	MINUTE		
1 1/2	1/2	24100	16.7	0.60	68
1 1/2	1	25600	17.7	0.56	65
1 1/2	1 1/2	39600	27.5	0.36	58

DISSOLVING TANK CAPACITY – 10 GALLONS
3/4" VALVE = 5 1/2 TURNS FOR FULL WIDE OPEN POSITION
WATER SUPPLY LINE PRESSURE – 72 POUNDS

FIG. 10. DRY FEED MACHINE; DISSOLVING TANK NO. 1

Tests were made in the dissolving tank to determine the efficiency for proper preparation of alum solution. For test No. 1, water was passed through the dissolving tank at the required rate of flow of 20 gallons per minute or 28,800 gallons per 24 hours equals 0.5 minute tank detention period, and the minimum plant rate of alum feed of 67.4 grams per minute or 214 pounds per 24 hours equals a 0.09

percent solution. The solution sample collected at the outlet of the dissolving tank showed that the alum was dissolved with water temperature at 55°F. The results of the laboratory tests for detention period of mixing required for floc formation with the various sized paddles operating at 0.91 F.P.S. peripheral speed, paddle length 50 percent of the bottle diameter, and using 73.0 cc. of the tank solution or 1.0 grain alum, in one gallon of raw water having 12 turbidity and 8.0 pH value, are shown in figure 11.

ALUM SOLUTION

DRY FEED MACHINE

DISSOLVING TANK NO.1 — WATER JET ACTION.

MINIMUM PLANT RATE OF ALUM FEED.

214 POUNDS ALUM PER DAY
28,800 GALLONS WATER PER DAY } = 0.09 PERCENT SOLUTION.

ALUM RATE = 67.4 GRAMS PER MINUTE.

WATER RATE = 20 GALLONS PER MINUTE.

73.0 CC. SOLUTION = 1.0 GRAIN ALUM.

PADDLE SPEED = 70 R.P.M.

DISSOLVING DETENTION PERIOD = 0.5 MINUTE.

ALUM SOLUTION TAKEN FROM OUTLET OF DISSOLVING TANK.

ALUM PER TEST = 1.0 GRAIN.

LABORATORY FLOC TEST — RAW WATER TEMPERATURE = 49° F.

PADDLE "A" INCHES	WATER TEMP. F. FINISH	MIXING PERIOD — MINUTES																			
		2	4	6	8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
1/32	54	N	N	N	N	N	N	N	N	N	N	N	N	F	F	F	G	G	G	G	E
1/16	54	N	N	N	N	N	N	N	N	F	F	F	G	G	G	G	E				
1/8	54	N	N	N	N	N	N	F	F	G	G	G	G	E							
1/4	54	N	N	N	N	N	F	F	G	G	G	E									
3/8	54	N	N	N	N	F	F	G	G	G	E										
1/2	54	N	N	N	N	F	F	G	G	G	E										

FIG. 11. ALUM FLOC TEST; LABORATORY MIXER. USING ALUM SOLUTION FROM DRY FEED-DISSOLVING TANK NO. 1. TEST NO. 1

For test No. 2, water was passed through the dissolving tank at the same rate of flow as in test No. 1, and the maximum plant rate of alum feed of 540 grams per minute or 1714 pounds per 24 hours equals a 0.71 percent solution. The solution sample collected at the outlet of the dissolving tank showed that undissolved alum was passing through the tank outlet with water temperature at 55°F. The alum also settles and collects in the tank corners where the water jet action is not effective for dissolving the alum.

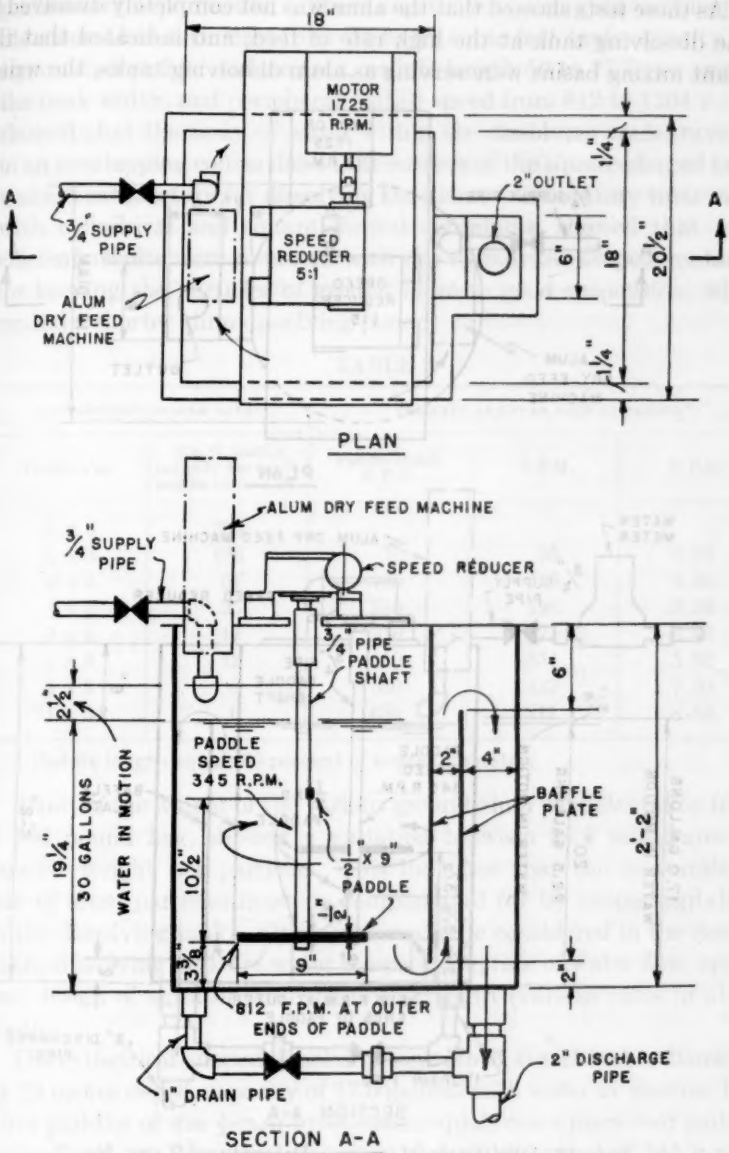


FIG. 12. DRY FEED MACHINE; DISSOLVING TANK NO. 3

As these tests showed that the alum was not completely dissolved in the dissolving tank at the high rate of feed, and indicated that the plant mixing basins were serving as alum dissolving tanks, the writer

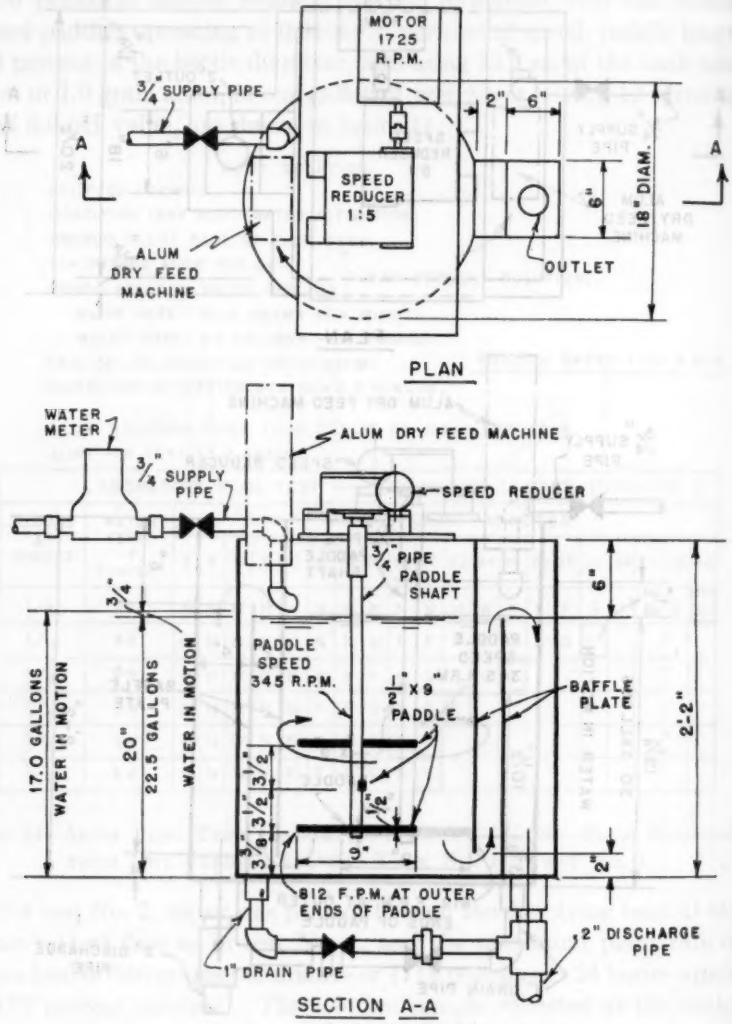


FIG. 13. DRY FEED MACHINE; DISSOLVING TANK NO. 2

designed and installed the cylindrical shaped dissolving tank No. 3 (figure 12).

A square shaped dissolving tank No. 2 (figure 13) was used for the

preliminary experiments. These experiments, using six various paddle assemblies with sizes from 23 to 83 cubic feet tank capacity per square foot paddle surface area, paddle length 50 to 77.7 per cent of the tank width, and peripheral paddle speed from 812 to 1264 F.P.M., showed that the body of water within the dissolving tank traveling in an overlapping course due to the corners of the square-shaped tank, was not satisfactory for dissolving the alum. Laboratory tests made with cylindrical and square shaped containers, showed that more efficient results were obtained with the cylindrical-shaped container for keeping the particles of ground alum in good suspension, which caused a shorter alum dissolving period.

TABLE 2

LABORATORY PADDLE DATA		VELOCITY AT OUTER ENDS OF PADDLE*		
Paddle size	Cu. ft. bottle capacity per sq. ft. paddle surface area	Paddle speed, R.P.M.	F.P.M.	F.P.S.
$\frac{1}{32}$ x 3	206			
$\frac{1}{16}$ x 3	103	70	55	0.91
$\frac{1}{8}$ x 3	51	150	118	1.96
$\frac{1}{4}$ x 3	25	250	196	3.26
$\frac{3}{8}$ x 3	17	350	275	4.58
$\frac{1}{2}$ x 3	12	450	354	5.90
$\frac{3}{4}$ x 3	8	550	432	7.20
1 x 3	6	650	511	8.52

* Paddle length equals 50 percent of bottle diameter.

A test of the weight of the various ground alum particles taken from a 200 pound bag, showed a variation between 24.0 milligrams to powder weight per particle. This indicates that the non-uniform size of alum particles must be compensated for by proper agitation in the dissolving tank. Other features to be considered in the design of the dissolving tank are water temperature, rate of water flow, speed and design of agitator, and solution strength (various rates of alum feed).

The cylindrical-shaped dissolving tank No. 3, size 18 inches diameter by 20 inches depth, capacity of 17.0 gallons with water in motion, has three paddles of size $\frac{1}{2}$-inch by 9 inches equals one square foot paddle surface area per 24 cubic feet water, and paddle speed of 345 R.P.M. giving a peripheral paddle speed of 812 F.P.M. with paddle length 50 percent of the tank diameter. The mixer assembly is operated by a $\frac{1}{4}$-H.P. motor connected to a speed reducer with a 1:5 ratio. For controlling the detention period of mixing in step with the various

water temperatures and alum feed rates, a meter is connected to the tank water supply line. Using 3423 gallons per 24 hours with the minimum plant rate of alum feed of 214 pounds per 24 hours gives a 0.75 percent solution, and the same amount of water with the maximum plant rate of alum feed of 1714 pounds per 24 hours gives a 6.0 percent solution. The tank detention period is 7 minutes.

It was noted that 8 times more water or 25,000 gallons per 24 hours, was used with the old dissolving tank No. 1 than with the new dissolving tank No. 3. This represents a saving of $1.00 (4 cents per 1000 gallons) for water per day when using tank No. 3 and also assures a properly prepared solution.

SUMMARY

1. Agitation is an important feature both in the dissolving box and the solution tank, for preparation of alum solution by the solution feed method.

2. Continuous mechanical agitation is the most effective method for dissolving ground alum in the dissolving tank.

3. Paddle position and speed, are important features for rapid dissolving action to keep the ground alum in good suspension.

4. For preparation of alum solution by the dry feed method, the important features of the dissolving tank with mechanical agitation are paddle size, paddle speed, tank capacity, and water supply control for the variable water temperatures.

5. A cylindrical-shaped dissolving tank, gives the best results for rapid dissolving action with mechanical agitation.

6. Prolonged high speed mechanical agitation of the alum solution in the preparation stage, has no preventive or retarding effect on the formation of alum floc.

7. Solution strength is not a controlling factor for alum floc formation with solution prepared by the high speed mechanical agitation method.

8. The dissolving tank for the alum dry feed machine using the high speed mechanical agitation method, should have a capacity of at least 5 minutes detention period, solution strength governed by the tank construction, a meter for the water supply line, and a peripheral paddle speed of at least 200 F.P.M. with paddle length 50 percent of the tank diameter and paddle surface area of one square foot per 5 to 15 cubic foot tank capacity with tank diameter the same dimension as the water depth.

(*Presented before the Illinois Section meeting, April 11, 1935.*)

THE DEVELOPMENT OF THE BREMERTON, WASHINGTON WATER SYSTEM

By Cliff C. Casad

(*Commissioner of Public Works, Bremerton, Wash.*)

The City of Bremerton owes its origin and growth to the development of the Puget Sound Navy Yard, the location of which the Federal Government decided upon in 1891 for the building of a dry dock and repair plants for maintenance of the battleships in the Pacific.

The city was incorporated in 1901, at which time it was a village of less than 300 persons and the city limits enclosed an area of approximately 350 acres. Since that time, the city has grown rapidly and with additional territory annexed or consolidated, the area of the city at the present time is about 1020 acres with a population estimated at 12,000. The territory immediately bordering the city is building rapidly and is occupied by an estimated population of 4,000.

When the city was incorporated there was no public water supply and one of the first steps of the city officials was to grant a franchise to one Harry Orchard to construct and maintain a system of water works to serve the city. The Bremerton Water and Power Company took over this franchise and started construction of the system with the source of supply at a small crib dam on Charleston Creek, a small stream about three and a half miles from the business portion of the city. The mains were of wood ranging from 4- to 6-inch in the distribution system and 8- to 12-inch mains laid for the supply line. This first unit was completed and water turned into the system July 4, 1902.

The supply of Charleston Creek was limited and to meet increasing demands for water due to the rapid building at this time, the water company raised the dam on several occasions to increase storage. When it was apparent that a further supply was necessary, the company secured a considerable area of land on the South side of Sinclair Inlet where Anderson Creek is located. About a half mile from the mouth of Anderson Creek the stream branches into what

are known as the East and the West Forks. A small dam was constructed on the West Fork of Anderson Creek and a 12-inch wood main was laid to the water and across the tide flat and Northerly to intersect the first supply line from Charleston Creek. This was completed in 1905.

In 1907 a dam was built on the East Fork of Anderson Creek. This is a crib dam faced with a veneer of concrete and provided a storage of about one and a half million gallons. An independent line was laid from this source of supply to the bay and followed the shore line into the city, thus providing the city with two sources of supply and independent supply lines.

The next step came in 1909 when it was necessary to install a pump on Anderson Creek about a quarter mile from the mouth of the stream. This creek is fed by innumerable springs and the stream at the pump location has a minimum flow of 1.5 million gallons daily or about one and a half times greater than at the location of the dams constructed above. The pump was operated by steam and with a capacity of 700 gallons per minute.

Following this installation came the construction of another dam on the West Fork of the creek. The location was about 100 yards below the first dam constructed in 1905. This added to the storage by approximately two million gallons and was completed in 1910. At this time the combined minimum supply was about one million gallons daily by gravity and one million gallons by pump.

In 1911 the company constructed a pumping plant on Gorst Creek. The pumping equipment consisted of a 90 h.p. gas engine with a triplex pump of 800 gallons per minute capacity. Gorst Creek presented the largest source of supply and to operate more efficiently, the pump installed on Anderson Creek was transferred to Gorst Creek in 1915.

In this year through an election the city took the initial steps to purchase the water system. After considerable court litigation the city took over the system in 1917, bonds being issued in the amount of $225,000 to cover the purchase price. Extensive improvements were then made under war-time conditions to meet immediate demands, instigated by Navy officials.

A concrete reservoir of 10 million gallons capacity was constructed conveniently near the first supply line laid into the city and this supply line was replaced with a 16-inch wood main which is now serving the city.

Additional extensions and replacements in the distribution system were made in 1918 and bonds in the amount of $175,000 were issued to take care of this expense.

As soon as electric power was available at the pumping plant, in 1918, a 10-inch 2-stage De Laval centrifugal pump, operated by 250 h.p. motor, with a capacity of 2600 gallons per minute, was installed, and in 1920 an 8-inch 2-stage De Laval centrifugal pump, operated by a 200 h.p. motor, with a capacity of 2000 gallons per minute, was installed. An emergency power plant, consisting of a 180 h.p. Sterling gasoline motor, was installed in connection with the 8-inch pump in 1921.

Prior to 1924 no effort had been made to obtain data concerning the water supply and in that year weirs were constructed on the various streams and master meters placed upon the supply lines. A 16-inch meter was placed at the pumping station to record the amount of pumping, and a 12-inch meter was placed on the supply line leading from Anderson Creek. Since that time we have accurate records of the flows of the various creeks.

Information received and compiled from weir readings of the creeks and from the master meters have proved very valuable in our study of plans for the future water supply for the city. With our knowledge gained from these records of the daily demands for water and of the supply available in the various creeks, either by gravity or by pumping, we have been able to get over some rather difficult situations when, due to dry periods, the supply was considerably diminished and on several occasions when necessary repairs to the pumps or motors curtailed the output to the city.

From our studies it was deemed necessary in 1928 to place a 4-inch pump of approximately 1,000 gallons per minute capacity on Anderson Creek near the location of the first pumping plant installed in the system. The wisdom of this installation proved itself when the large motor at the Gorst Creek pumping plant was disabled for a week in July, 1928. Demands for water at this time were very great and the service from this small pump installed on Anderson Creek removed the danger of a serious shortage which threatened not only the City of Bremerton, but also the Puget Sound Navy Yard which is dependent on water supply of the city.

Further study of our pumping system showed the advantage of a more elastic arrangement in pumping, and a 6-inch single stage pump with 150 h.p. was added to the Gorst Creek plant in 1929. Tests of

this unit showed a valuable addition of the pumping capacity of approximately 1900 gallons per minute.

In 1929 special water works bonds of $90,000 were voted. The dams on Anderson Creek were replaced and new mains were laid from the dams to connect with the supply line leading into the city along the water front. In addition, approximately 4500 feet of the old 16-inch line leading from the pumping plant were replaced with 24-inch steel arc welded pipe. This work was done by contract and was the last improvement to the water system which the city let out by contract. Since that time all improvements have been done by day labor.

In 1929 at the beginning of the depression Bremerton was hard hit as were other communities throughout the nation. The Navy Yard, which had employed approximately 4,200 men until the fall of 1929, suddenly dropped some 1300 employees from its rolls. These people had been called to work in the Navy Yard from all parts of the country and when they found themselves without employment presented a serious task to relieve the distress caused by their losing their jobs.

RECONSTRUCTION OF SYSTEM

The physical condition of the water system as a whole was such that a great deal of work could be done to advantage on the system, and, accordingly, a program of replacement was mapped out to cover a period of ten years. The system was practically entirely of wood pipe and very few replacements had been made since the original construction. A chart was made of the various lines in the distribution and supply system, the dates of original construction noted, and a schedule of replacements worked out to commence with an allowance of 20 years time from the original date of construction, thus automatically placing the date for the replacement.

Because of the greater distress in the winter months, the city has followed this plan yearly and provided a great deal of employment during this period.

Due to the proximity of the Navy Yard, the city is fortunate to have available almost every kind of skilled labor and rarely needs to employ outside labor on any of our projects.

This plan of replacing the system by reconstructing certain units each year is of particular advantage to work necessary on the supply system. Fortunately, the city has two supply lines leading into the

city and during the winter months, when consumption is low, it is possible to put one of the lines out of commission for a period of time while replacement work is being done.

Two units of this work have been added to the main supply line from Gorst Creek pumping plant which was laid under contract in 1930 and approximately 6,500 feet of the 24-inch main has replaced the 16-inch main over a stretch which is particularly difficult and costly to maintain. There remains about 12,000 feet of this supply line to replace, of which our schedule calls for 3,000 feet to be made this coming winter.

To date approximately 15 miles of cast iron pipe have been laid in the distribution system in replacement of wood mains. There remains 32 miles of wood pipe in the system.

Gate valves are placed at all intersections to afford a minimum of discomfort to consumers when it is necessary to shut off the pressure. Only a few years ago it was necessary to shut off the entire business portion of the city in order to repair a bad break in the wood main which has since been replaced with cast iron. On numerous occasions, until recent years, it was necessary to shut off territory to the extent of forty acres to make needed repairs to certain sections of the system. Every effort is now made to eliminate dead ends.

Copper services are now used for all connections to cast iron pipe and replacing the old galvanized service pipes whenever wood mains are replaced.

FINANCING REPLACEMENTS

The cost of replacements has been borne by the Water Department and the finances of this department have not always been in such shape as to carry on a considerable program. In 1925 a large paving project necessitated the replacement of considerable mains. This replacement marked the beginning of the laying of cast iron pipe in the system.

Finances of the Water Department were such that taking care of this work was a problem. There were not sufficient funds on hand, revenues were not sufficient, and voting of a bond issue was out of the question. As the paving was constructed under the local improvement plan, an idea was conceived to borrow funds from the Local Improvement District and to assess this amount back to the Water Department and to retire the assessment in eight years as provided for in the plan of payment to the property owners. Over $33,000

were expended in the improvement in this manner. This method of financing the extensions and replacements was discontinued upon the advice of the State auditor, but had given the Water Department the financial breathing spell it needed and made it possible to do work which otherwise seemed impossible. This obligation has since been removed and the department was benefited by a very substantial improvement to its distribution system.

Finances in 1930, when the replacement program of water mains was started, were still at a low ebb. Monthly revenues were sufficient to care for payrolls of the work program which we had laid out and the city contracted for the purchase of cast iron pipe to be paid for in installments over a period of several months. When material was delivered the plan was worked out to provide a maximum number of persons with employment through a weekly revolving of as much labor as possible. It was planned to complete our reconstruction by April 1 when activities in the Navy Yard offered opportunity for other employment. About this time the Water Department was considerably in the red due to these activities and, with the cessation of construction work, the revenues of the department were allowed to accumulate so that by September 1 the Water Department was again on a cash basis. By the time we were ready to start our next period of work we were financially able to take care of the preliminary expense and the process was repeated.

In this manner for the past five years we have been able to reduce materially the distress caused by unemployment. Approximately $165,000 were spent on the water system.

TREATMENT

The city is fortunate in having for its largest consumer the Puget Sound Navy Yard which uses an average of 750,000 gallons daily. Through the chemical and medical laboratories in the Navy Yard the water is constantly being checked for quality and no complaints have been registered.

The city is always seeking to better the quality of the water supply. Settling basins and channel control to permit sedimentation have greatly improved the supply. This is very noticeable as compared with former years when cleaning the reservoirs.

All water is treated with chlorine and in addition the water of Gorst Creek is treated with ammonia.

TASTES AND ODORS

It may be of interest to some of the members to know our experience from the taste resulting from the combination of chlorine in creosoted wood pipe. The line extending from the reservoir on Anderson Creek which was constructed in 1930 is creosoted wood pipe. Water was allowed to flow through this newly constructed pipe for two weeks and wasted into the bay in an effort to wash out the effects of the creosote on the water. When the line was finally opened for service there were a great many complaints regarding the taste. The water was treated with chlorine at the intake and the combination of chlorine-treated water with the creosote in the pipe produced a very disagreeable taste. It was demanded that the city discontinue the use of this line. As the period was midsummer and demands for water great, the approximately million gallons daily received from this source was considered as being necessary, and as it was a gravity supply its discontinuance and the additional cost of pumping from Gorst Creek plant would mean considerable expense.

Upon investigating what might be done I learned that the application of ammonia in an amount of about two pounds per million gallons made to the water before it was treated with chlorine would take away the disagreeable taste. As we were not equipped to make the application of ammonia it was decided to try an experiment with concentrated ammonia which I purchased at a local drug store. A gallon of concentrated ammonia, containing about two pounds, was mixed with nine gallons of water in a 10-gallon keg. The spigot was arranged so as to discharge its contents at a rate sufficient to empty the keg in 24 hours, and the disappearance of the disagreeable taste was noticeable in a few hours thereafter. Our experiment was considered successful.

However, a bad forest fire on the water shed took up the attention of the man responsible for keeping our apparatus charged with ammonia and he had to neglect it for several days causing the return of the disagreeable taste. After the treatment was resumed we had no further complaints.

WATER WASTE

The work of replacement during the past five years very greatly reduced the amount of waste as indicated by the master meters on the supply lines. An effort was made to check up various sections of the system where replacements were contemplated. Methods used

consisted of shutting off the district desired to be tested and connecting two fire hydrants, one being in the particular district and one being on a live line, a meter placed in the hose connections between the hydrants and the water turned on and both hydrants then opened. These experiments were made between 2 and 4 o'clock in the morning when consumption was practically nothing, checks being made at any meters where continued service was necessary.

It was surprising to learn that leaks amounting to as much as 90,000 gallons daily were found, but owing to the porous character of the soil the water was permitted to sink into the sewers without being detected upon the surface.

LAYING CAST IRON LINE INSIDE OLD WOOD MAIN

During the last year of our replacement operations it was necessary to replace a pipe line under a paved street. The line to be replaced was a 12-inch wood main and after some study of the situation it was determined to lay this main without breaking some 1200 feet of concrete. Adjoining streets were gravel surfaced and offered an opportunity to by-pass the 12-inch line which we proposed to replace. Examination of the old line showed it to be laid to very good alignment and grade and it was decided to lay the 6-inch cast iron line inside of the old 12-inch wood line. Openings were made at street intersections and at 300 foot intervals, cable from a hand hoist was threaded through the main and a special shoe made for the spigot end of the first length of 6-inch pipe. The cable was secured to the hub end of the pipe and the pipe was drawn into the wooden main, spigot end ahead, until room was made to caulk the next length, after which the process was repeated. The job was completed with a minimum of inconvenience to the travel along the street and at considerable saving by avoiding the breaking of the concrete pavement.

WATER USE

The results of our work during the past ten years are well illustrated by the yearly consumption from gravity and pumping as indicated by the master meters. In 1925, 890 million gallons passed through the meters as compared with 1929 which was the peak year with 1,056 million gallons, and 714 million gallons in 1934, indicating the past year to be 176 million gallons less than in 1925 and 342 million gallons less than in 1929.

There were 4,024 active accounts on January 1, 1935. Of this number 704 have been added since 1929 showing a very healthy growth during the past five years. We are about 90 percent metered.

The city is making every effort to safeguard the future of the water supply through securing all the available water shed of the present supply, the securing of water rights on other streams, and negotiating for the necessary water shed of these streams from timber companies now operating on the lands. Logged-off lands on these water sheds will be re-forested and vegetation encouraged to replace the barren areas. These additional water supplies will not be needed for some time, but with the normal growth of the community, by the time the water sheds have regained a covering of vegetation and the run-off has again increased to its present volume, they will fit very nicely into the needs of the future supply for many years.

(Presented before the Pacific Northwest Section meeting, May 17, 1935.)

DOMESTIC WATER SUPPLIES OF IDAHO

By W. V. Leonard

(State Chemist and Sanitary Engineer, Boise, Idaho)

We have at the present time 82 domestic water supplies in the state of Idaho. Fifteen of these are owned and operated by private companies. The remaining 67 are owned and operated by the cities and villages. A list of the sources of these waters is of interest in showing the probable types of water to be furnished to the citizens.

Open streams and lakes are used for domestic water supplies in 19 cities and towns. Four of these use lake water and 15 open streams. Of these all are subject to intermittent contamination from colon bacillus and other micro-organisms.

Of the 19 surface supplies referred to above, 14 are chlorinated. Four have regular filtration plants in daily operation. Five do not safeguard the supply other than care at the division point in screening the intake. There is a direct relation between these plants and the effect of stream pollution. Idaho is blessed with supplies of springs originating in higher altitudes whose source is only a few miles from the towns using these waters. Twenty-three of the villages use springs as a water supply. Due to the fact that we have a limited population few of whom live at the present time above the springs, this source of water supply has been favorably received. Constant vigilance and care may keep these springs from becoming grossly polluted.

WELLS

Forty of the towns and cities of Idaho have wells from which the water is pumped. Six of these use chlorination as a safeguard against intermittent contaminations. In the lava formation of southern Idaho wells are constantly being condemned for use as domestic water supplies unless sterilized. The fault formation of the lava allows underground pollution from sewers, and waste water, as there is no filtering through this lava, which is full of caverns, flow holes and direct fissures to the surface. It is readily seen that this underground body of water may at any time become laden with undesirable or pathogenic bacteria.

HARDNESS

The domestic water supplies of Idaho would be classified as medium to soft. Snake River in its progress through the state "picks up"

additional calcium and magnesium compounds through return flow and has a maximum hardness of 250 p.p.m. at the period of low flow. The northern part of Idaho using lakes and surface supplies has very soft water, the average being around 50 p.p.m. hardness. The City of Buhl is at the present using slow sand filters after sedimentation. This has not proven to be a highly successful plant, because algae in the supply have a tendency to clog the filter and consequently very short filter runs result.

FLUORINE

In certain sections of our state the deep seated artesian water carries in solution compounds of fluorine.

Fluorine in certain concentrations has an injurious effect upon the enamel of the teeth. In certain localities practically 100 percent of the school children born and raised, in home and schools, using this water have developed this enamel deficiency.

The problem of the fluorine compounds is of the greatest importance in some localities of our state. In one town in southeastern Idaho the Sanitary Engineering Division recommended that the village acquire the local water company and discard the supply as it contained fluorine in amounts sufficient to cause dental deficiency in all of the children. This was done and after several years use of the new supply no dental troubles are apparent.

In any comprehensive planning for cities in Idaho, there can be no more critical feature to be looked into than the supply of domestic water.

A safe drinking water is paramount to all other considerations and as we are deeply interested in all things that concern the public health, the constant supervision of plant operation and records of analysis of the water supply is of extreme importance. Through skilled engineering practice some of our cities have taken raw water of a very highly polluted nature and through additions of chemicals have made a safe, sparkling water, which is life to man and animal.

The standard of purity as established by the Treasury Department of the Federal Government is used in Idaho.

The Water supply shall be: (a) obtained from a source free from pollution, or, (b) obtained from a source adequately protected by natural agencies, or, (c) adequately protected by artificial treatment.

(*Presented before the Pacific Northwest Section meeting, May 17, 1935.*)

ECONOMIES IN THE SELECTION AND OPERATION OF ELECTRIC POWER FOR PUMPING EQUIPMENT

By D. P. Minichan

(*Engineer, Appalachian Electric Power Company, Roanoke, Va.*)

Most of us are conscious of the fact that cheap electric power may be made possible by allowing water to run from a higher to a lower level through a water wheel connected to an electric generator. It is just as simple and as logical to reverse the process and pump water cheaply by raising it from a lower to a higher head pressure by means of a water wheel driven by an electric motor.

There has been a marked trend in the last few years toward the use of electric power for pumping water. Some of the factors which have influenced this trend are as follows:

(1) The improvement in the net-work of power lines has created a reliability of service that makes shut downs a rarity and then but momentary.

(2) Centrifugal pumps have been developed with such high efficiencies that purchased electric power compares favorably with the most efficient steam pumping machinery.

(3) Electrically operated pumping stations are clean, neat, and inconspicuous, thus permitting their location within residential districts.

(4) Electrically equipped stations require a small personnel for operation and maintenance.

(5) Centrifugal pumps require little repair work and the electric motor scarcely any attention except periodic lubrication.

(6) Electrically equipped pumping stations lend themselves to automatic or remote control, so that it is feasible to build stations of relatively large size which require no attendance except for periodic attention.

The selection of the proper type of pump for a given condition is not an easy task. Anyone who is given this responsibility should be guided by the latest engineering practices and a study of the conditions the pump is to meet. These conditions should be as carefully compounded as a physician's prescription, and the pump selected

should fulfill the prescribed conditions. In making a study of the conditions, the engineer should consider these facts.

(1) Number of units required.

(2) Capacity of each unit.

(3) Total lift including suction lift and pipe friction.

(4) Suction lift, distance from supply, size of suction pipe and number of bends.

(5) Variation in lift, if any, for both suction and discharge.

(6) Quality of liquid, whether clear water, gritty, acidulous, solids in suspension, etc.

(7) Temperature and viscosity of liquid.

(8) Minimum head unit will have to operate against.

(9) Is service continuous or intermittent.

No pump should be purchased without a thorough consideration of its operating characteristics as applied to the head conditions at the location at which it is to be installed. Stock pumps, although often desirable for different classes of service, are more often undesirable from the point of view that the head design is not fitted to every particular head condition. Under all conditions, a careful engineering check should be had prior to the installation of any pump.

TYPES OF PUMPS

For all practical purposes, pumps may be divided into two general classes, namely; displacement pumps and centrifugal pumps. In all displacement pumps the volume of liquid pumped is always less than the piston displacement, or in the case of rotary pump the displacement of the rotating element. This difference is due partly to leakage past the piston or rotating element and partly due to leakage through the valves. This difference or "slip" may vary from 2 to 50 percent depending on the age and condition of the pump.

Most displacement pumps are of the reciprocating type with piston speeds of from 30 to 80 feet per minute. These low piston speeds in turn require extremely low shaft speeds as compared with ordinary motor speeds. This condition is met by installing a slow speed, high starting torque motor with belt or silent-chain drive, by a gear reduction or a combination of the two. This reduction in speed, especially when accomplished by gear reduction, necessarily involves a considerable loss of power, and this loss should be considered in comparing pump efficiencies.

The majority of pump installations now being installed are centrif-

ugal pumps directly connected to horizontal or vertical motors. The centrifugal pump is one in which curved vanes or impellers, rotating inside a close fitting casing, draws in the liquid at the center and by virtue of centrifugal force, throws out the liquid through an opening at the periphery of the casing. This centrifugal force causes the water to assume a kinetic energy which forces it outward and upward against a given head, this head depending on the peripheral speed of the particular impeller.

The centrifugal pump is one of the most widely used types of pumping machinery and it is without doubt the one about which least is known. This applies to the operation as well as to the selection and it is only in comparatively recent times that this method of pumping has been efficiently applied to pumping problems. The difference between pumping and pumping efficiently is a point which was not emphasized. The principal consideration in design and manufacture was low first cost and almost everything was sacrificed to this end. This has all been changed; today the centrifugal pump is one of the most efficient pumps made. Centrifugal pumps are now designed for the highest possible efficiency, and, what is more important, every unit should be selected to operate at its maximum efficiency under the conditions for which it was intended. A centrifugal pump has certain fixed characteristics, which are determined when the pump is designed and which the purchaser cannot readily alter by adjustment. Likewise, the motor or other driving agent has exact characteristics which ought to be adapted to the pump.

TYPE OF POWER

The engineer who is confronted with the problem of choosing the most suitable type of power for driving pumps has two main factors to consider in making his selection. These factors are the initial cost and the operating cost of the equipment. The operating cost of an electrically driven pump is a known and definite quantity, depending on its efficiency, and compares favorably with the operating cost of any other source of power. From the initial cost standpoint electrically driven pumps are superior to any other source of power for the following reasons:

(1) Original cost of equipment is less.
(2) Motor foundations cost less than engine mountings.
(3) Smaller building is required.
(4) No fuel storage is necessary.

(5) Lower insurance rates are obtainable.
(6) Lower maintenance cost.
(7) No fuel transportation charges to consider.

The contingency element of breakdown should be given due weight, and dangers sustained as a result of breakdown periods should have a qualifying effect on the decision as to the type of power to select. The electric motor due to its simplicity of design and rotary motion is singularly free from mechanical troubles when properly fitted to its load.

On account of the peculiar characteristics of the centrifugal pump, special care is required in selecting the type of motor best suited. With a reciprocating pump operating at constant speed an increase in the head pressure increases the load on the motor, but with a centrifugal pump an increase in head pressure reduces the motor load. The volume of water delivered by a reciprocating pump is not affected by the reduction of the head, but the power required is reduced. A reduction of the head with a centrifugal pump, however, increases the volume of water, and as the efficiency at the same time goes down rapidly, the load increases.

The performance characteristic of a centrifugal pump depends on the design of the pump runner or impeller. For a change of speed the capacity will change in direct proportion to the speed change, the head will change in direct proportion to the square of the speeds, and the horse power in direct proportion to the cube of the speed. From these facts we may conclude that, if a centrifugal pump is to operate efficiently, it must operate at or near its design head pressure and at its design speed. No type of drive is so well adapted to drive centrifugal pumps as the constant speed electric motor.

The centrifugal pump is ideally adapted for electric motor drive in view of its requiring a low starting torque, resulting in comparatively small rush of current in starting, so that standard squirrel-cage motors can be used even of high horse power.

The speed of centrifugal pumps is higher than can be used for displacement pumps; hence cheaper motors can be used and there is an absence of gear reductions. In well designed centrifugal pumps, there is little danger of overloading the driving motor, so that motors can be selected close to the rating of the pump. This results in high efficiency and high power factor of the motor.

The squirrel-cage induction motor is the most popular electric power unit for driving pumps, because of its simplicity of construc-

tion and adaptability to widely varying duties. Induction motors are available in any desired rating and are so constructed that they have high strength and rigidity, yet low weight per horse power. There are many possible mechanical variations, such as totally inclosed, splash proof, flange or vertical mounting, and ball or sleeve-bearings. In addition, squirrel-cage motors with special electrical characteristics, such as high starting torque with low starting current current and normal starting torque with low starting current, are available in standard ratings.

Another type of electric motor which is ideally suited for water pumping is the synchronous motor. Synchronous motors differ from induction motors in several respects, one being that the magnetizing current of a synchronous motor is not obtained from the power circuit, but by means of a direct-current excited field on the rotor of the motor.

The application of synchronous motors is not so well understood as that of induction motors and, therefore, it would be well to discuss some of the more pertinent factors as regards characteristics, field of application and types available.

Whether or not a synchronous motor is chosen depends on an evaluation of the following economic factors:

(1) Operating efficiency.
(2) Operating power-factor.
(3) Constant speed.
(4) Space requirements.
(5) Initial investment.
(6) Service facilities.

The efficiencies of synchronous motors are uniformly high. In general, they are higher than those of any other type of constant-speed motors, although, in the smaller horsepower ratings, the fundamental design characteristics of synchronous motors tend to limit this advantage. A typical example of the advantages of high operating efficiency is shown in the following: A two percent gain in efficiency in a 200 horsepower motor is approximately equivalent to a 4-Kw. reduction in losses, which, based on 3000 hours operation per year, 29 percent load factor at 1½ cents per Kw-hr. is equivalent to $180.00 per year savings in the power bill.

The cost of delivering electric power to a station depends partly on the power factor of the electrical load. Most power companies have "power factor clauses" in their power rates. These clauses

reward the customer for high power factor operation by allowing certain credits on the customers' power bill. Since synchronous motors operate at or near unit power factor they are very desirable as a means of lowering power costs.

Synchronous motors operate at constant speeds at loads within their capacity. On pump applications, this is a distinct advantage since maximum output is obtained from the pump.

Public-works pumping provides an ideal application for synchronous motors, since a large part of the load in such service is constant and continuous.

It is on those applications which operate, in many cases, twenty-four hours a day, that the increased values of power factor and efficiency have very material effects on the cost of power.

Low speed synchronous motors are suitable for direct connection to large-volume, low-speed, low head pumps, thus eliminating gear units and making possible the attainment of high efficiency and unit power-factor. High speed synchronous motors may be direct-connected to high-speed, high-head pumps.

Experience has taught us that 95 percent of municipal pump applications can be covered by a single stage, double suction, split casing pump with either a squirrel cage or synchronous motor. Squirrel cage motors are universal below 50 H.P. and as a rule will be 220 or 440 volt. From 40 H.P. up, 2200 volt motors are obtainable. Above 50 H.P. and especially at low speeds the synchronous motor is more desirable, especially where high power factor is rewarded.

STATION CONTROL

One of the most advantageous factors of electrically driven pump stations is the flexibility of electric control. Stations may be designed so that they are manual, semi-automatic or full-automatic controlled. The manual station consists of the proper starting switch with the necessary motor protection. The motors, however, must be started and stopped by hand. The full automatic station will operate under certain predetermined conditions without the attention of an attendant.

Pikesville, Baltimore, Station

An outstanding example of a full automatic pump station is the Pikesville automatic station in the City of Baltimore. This station is located underground and is equipped with two pumping units.

One of these is selected each week to be the pumping unit and the other acts as a stand-by unit. The full-automatic control provides for the starting and stopping of the selected pump in response to changing pressure demands, and also for the starting of the standby unit in case the selected unit is thrown out of service. Once each week the station is visited by an attendant for the purpose of changing charts, changing the selected units and making other routine adjustments. Because this station is unattended is no reason the supervisor does not know what is going on. For he may call from any telephone at any time by dialing the pump station number and receive an audible indication as to the suction and discharge conditions, and a complete report of the operating condition of the mechanical equipment.

Another example of the flexibility of electricity for driving pumps is the constant pressure variable speed pump. Nearly every large water system presents the problem of maintaining constant delivered water pressure. Too low a pressure may cause failure of water in parts of the system; too high a pressure increases pumping and maintenance cost. This trouble may be overcome by the use of a variable speed motor connected to a centrifugal pump. The pump control is actuated by two pressure regulators one set for high pressure and one for low pressure. If the system pressure falls below a certain setting the low pressure regulator operates and causes the motor to run at a higher speed. After the system pressure has been restored, the high pressure regulator operates and causes the motor to run at reduced speed. This system maintains constant pressure at a minimum cost.

In selecting pumping units, it is often thought that the pumping units should be built for some anticipated future condition, which may or may not develop. In most cases it pays to buy the pump for the conditions existing at the time and if conditions change, replace the pump with a more suitable one. If future needs must be considered, it is better to purchase the pump for present conditions with large enough casing to accommodate a larger capacity impeller and the motor large enough to take care of the anticipated condition. If this is done, the pump impeller alone will have to be replaced while the motor will operate almost as efficiently at the reduced load.

From the foregoing, we may conclude that an electric motor with suitable control can be designed to take care of any pumping condition regardless of its complications.

DISCUSSION

E. F. DUGGER (General Manager, Water Works Commission, Newport News, Va.): Mr. Minichan's paper gives a most clear and comprehensive discussion of economics in the selection and operation of electric power for pumping equipment.

There is and has been for the past few years quite a decided tendency towards electrifying municipal pumping stations, chiefly for the reasons outlined by the author of the preceding paper. Along with this tendency to electrify pumping stations there has been the increased use of centrifugal pumps. This type of pump has well encroached upon the field generally requiring reciprocating types. Not only have sizes and capacities of centrifugal pumps been materially increased, but their application has been extended to higher heads which only a few years ago could have been attained by the reciprocating pump.

Developing has rapidly continued also in motor pumps, consisting of a motor and pump built as a single unit, thus being a compact, light weight unit which requires no base plate.

The reciprocating pump has not lost its place and still holds its own where flexibility in operation under widely varying head and capacity is demanded. However, the higher speed centrifugal pump has naturally fallen in line with the electric motor drive.

The author states that the centrifugal pump is economical in operation only at or near the designed point. However, the writer notes the tendency of some pumps to have a rather flat head capacity curve, which marks another encroachment on the field of reciprocating pumps.

At the plant which I operate, we have one small capacity centrifugal pump in the same system with two pumps of a larger capacity. Thus by single or combined operation, we can operate at a reasonably high point of efficiency.

I wish to emphasize the point brought out by the author with regard to the use of stock pumps in water pumping stations. The water plant, as no other pumping plant, has constant service in which pumps must supply the demand continuously. It takes but little figuring to see that it is well worth the little extra money to purchase a pump especially designed for this specific service desired.

Every responsible pump manufacturer maintains a laboratory in which he continuously works to improve his pump, and in which he can test the completed pump or a model of the same. With this

service available it is only the part of wisdom to get the pump characteristics directly from the manufacturer.

It is likewise a simple task to run a test on the installed pump to determine characteristics under actual working conditions. Such a test not only serves to check laboratory data, but familiarizes the operator with the pump characteristics.

Such first hand information to the operator or even information which he has had some part in securing encourages that operator to run his pumps at a point of maximum efficiency whenever possible.

The selection of the motor, as that of the pump, should be as carefully made as possible. The ordinary squirrel cage motor is the simplest, and most rugged type of all, and I have found that for large pumps that are operated for a large part of time, the synchronous motor is the proper drive due to the high power factor obtainable.

LEE HALL STATION

Until November 15, 1934, the pumping equipment at the main or Lee Hall Station of the Newport News Waterworks Commission consisted of the following steam equipment:

3—3,000,000 gallon horizontal triple expansion double action steam pumps.

1—1½-million gallon of the same design, complete with auxiliaries which varied from 30 to 15 years each.

Due to the age and natural depreciation of this equipment it was necessary either to completely overhaul these units or to purchase entirely new equipment to insure dependable service and higher degree of efficiency.

After a careful and complete study, it was decided to consider completely new equipment. The final decision was to install electrically driven centrifugal pumps using purchased power. The standby unit was a Sterling Viking, 6 cylinders, rating at 300 B.H.P., at 1200 R.P.M., and was installed to fully conform with National Board of Fire Underwriters' requirements.

The new equipment consists of three hydraulic pumps, two at a rating of 3,000, and one at 1500 g.p.m., 1800 r.p.m. against the total head of 220 feet. Motors of 200 horse power each are provided for the 3,000 g.p.m. pumps.

Directly connected to each of the 3,000 g.p.m. pumps is a 200 horsepower, 200 volt, 3 phase, 60 cycle, synchronous type motor for a continuous rating of 1.0 P.F., 40°C, Starter and field, and equipped

with a direct connected 125 volt D. C. shunt wound excitor. The 1500 gallon per minute pump was directly connected to a 100 horse power motor of the same characteristics. Both pumps and motors were manufactured by the Allis Chalmers Manufacturing Company.

Three low head pumps were also installed in this plant, two of which were 3,000 g.p.m., at 865 r.p.m., against a total head of 30 feet. The 1500 gallon unit at a rating of 1160 r.p.m. against 30 foot head.

The switch board for the control of the entire plant was manu-

TABLE 1

UNIT NUMBER	1			2		3		
	Mfrs. guar.	Mfrs. test	Field test	Mfrs. test	Field test	Mfrs. guar.	Mfrs. test	Field test
High head pumps								
R.P.M.	1800	1800	1803	1800	1802	1800	1800	1798
G.P.M.	3000	3000	3030	3000	3060	1500	1500	1510
Total head, feet	208	219	223.4	220	220.9	208	210	221.6
B.H.P.	200	201	197.1	203	199.0	100	100	98.9
Efficiency, per cent*	85.5	85.3	86.5	85.8	86.0	85	84.7	85.4
Low head pumps								
R.P.M.	865	865	868	865	869	1160	1160	1160
G.P.M.	3000	3000	3050	3000	3020	1500	1500	1450
Total head, feet	30	32.8	29.8	33	30.5	30	32.8	29.4
B.H.P.	30	32.1	26.6	33.1	27.4	15	15.9	12.6
Efficiency, per cent*	85	87.8	86.5	87.4	85.1	85	89.5	85.5

* Mfrs. test, based on shop test curved and guaranteed motor efficiency; and field test, based on guaranteed motor efficiency.

factured by the General Electric Company and consists of five panels. The operation of the motor starting panels is manual with push button control. All motor starters are of the full voltage type.

A brief description of the operation of the motor panel is as follows:

Pressing the button on the control panel causes the line contractor to close applying full voltage to the motor. As the motor accelerates to near synchronous speed, the slip frequency field applying relay closes and causes contactor to close, and apply the field at proper time.

This entire installation was completed, tested and accepted in November 1934, and each of the units exceeded the guarantee.

Table 1 shows the manufacturer's guarantee overall efficiency of the pumping unit, and results obtained by actual field test.

The total cost of the entire change over was approximately $60,000-000. The operation of the new equipment was such that the cost of the new equipment has exceeded the estimated saving for the first ten months operation. The total economies as a result of this work have amounted to approximately $10,000.00.

From a copy of our last power bill shown below it may be easily seen that with the rate we have and with the high efficiency of the unit, it is certainly a profitable operation to pump by electricity instead of with steam with the usual operating efficiencies.

For electric service furnished for power from 8/31/35 to 9/30/35 as follows:

To 8,000 Kilowatt hours at 0.015	$120.00
40,000 Kilowatt hours at 0.01	400.00
93,200 Kilowatt hours at 0.008	745.60
141,200	$1,265.60
Demands (demand 226.17 KW at $1.50)	339.25
288	$1,604.85
234 (less 7½ discount)	120.36
232	$1,484.49
3)754	
251.333 (Coal Adj. 141,200 KWH × .00031)	43.77
Power factor 100 percent	$1,528.26

Meter read 4012 KWH, 9/30/35
Meter read 3659 KWH, 8/31/35
Consumed 353 × 400 = 141,200
Cost of coal $4.81

(Presented before the Virginia Section meeting, November 7, 1935)

DIESEL ENGINES IN THE WATER WORKS FIELD

By W. S. Russell

(Sales Engr., Bluefield Supply Company, Bluefield, W. Va.)

The so-called Diesel engine is an internal combustion engine which ignites its fuel due to the heat generated by compressing air. In the conventional four cycle Diesel engine, the sequence of operation is as follows:

1—On the power stroke, the piston is driven down by pressure of the exploded gases.

2—On the following upstroke, the burnt gases are expelled through the opened exhaust valve.

3—On the following down stroke, intake valve opens and the piston sucks in nothing but air.

4—On the following up stroke, the air is compressed to a pressure of about 500 pounds per square inch, which raises the temperature to approximately 500°Fahrenheit. At the top of the compression stroke, the fuel charge is injected into the highly heated air, the temperature of which is sufficient to explode the fuel. Fuel injection is usually accomplished by the use of individual piston type pumps which spray the fuel charge into the cylinder under such tremendous pressure that it is practically atomized.

This different method of handling and combining the fuel and air charges allows the engine to operate on a fuel which could not possibly be carburreted successfully. We find therefore that this system allows the engine to burn what are known as fuel oils and low grade distillates very successfully. These are materials which for years have been considered almost valueless by-products from the petroleum cracking processes employed in the manufacture of gasoline and benzine etc. The economies which result from the operation of Diesel engines, therefore, are due to the following causes:

1—Operation on a low cost fuel which in some parts of the country can be bought for as little as 3 cents per gallon.

2—The higher B.T.U. content of these low grade fuels, which may run to about 18,000 B.T.U. per pound as against about 14,000 B.T.U. per pound for ordinary gasoline.

3—The higher thermal efficiency of the Diesel engine. By this we mean that the Diesel engine may convert as much as 35 percent of the available heat energy into mechanical energy, while the gasoline engine will convert only about 23 percent of the available B.T.U. content into mechanical energy when operating at peak efficiency.

These facts of operation have more and more turned the attention of the public to Diesel engines when it is desired to effect economies in the operation of engine driven equipment.

During the past several years, the rapid improvements in the design of Diesel engines, coupled with the reduction in prices which has been accomplished due to larger scale production, have made these units increasingly popular as prime movers in the water works field. Up until several years ago, the Diesel engine was still largely in the experimental stage, but during the past five years this power unit has definitely come out of the woods and taken its place along with the other prime movers as a factor to be considered seriously before deciding upon power to be employed.

The Diesel field has been considerably broadened of late by the development of small Diesel generator sets at prices much lower than those of the large so-called stationary heavy duty engines. So far, this type of high speed Diesel has found its most useful application as a stand-by unit, but the manufacturers of this type engine are making greater and greater claims for these engines all the time.

In the water-works field, the Diesel engine has proven itself to be a most adaptable unit. It is used in combination with generator sets to produce standby power in case of power failures on the regular source. It is used in the same manner as a central generating plant to furnish electric current to scattered booster stations throughout a municipality or water district. Again, we find the Diesel direct coupled to a deep-well turbine, operating an air compressor in conjunction with an airlift pump, or operating a centrifugal pump through a speed increaser or a conventional belted drive.

It is, of course, hardly necessary to say that under conditions of equal cost, any utility operator would prefer to use electric current furnished from a never failing source by a reliable power company whose representative appeared to read the meter once a month and

is then seen no more for another thirty days. Unfortunately, however, this condition is not always obtainable. In some cases, it may be that the high power rate prevailing drives the utility operator to the installation of Diesel power. In others, it is the unreliability of service or restrictions on the hours of operation imposed by the power company, or electric current is simply not available. In some instances, utility operators have installed their own Diesel plants either out of spite to the power company or through an honest but mistaken belief in the ability of such an installation to effect impossible economies. The old saying that figures do not lie, but liars will figure never had a more pertinent application than in the field of figuring operating costs per unit of power developed by a Diesel engine. The writer has often seen the same set of operating conditions analyzed by four or five engineers, all of whom were figuring on installing substantially the same type of equipment at closely comparable prices, with total operating costs varying all the way from 4 mills to 1.1 cents per kilowatt hour. Computations of this sort, arrived at by irresponsible or inexperienced engineers, have frequently been responsible for installations of Diesel equipment in the belief that economies to be effected by the installation would soon pay for the equipment.

At first glance it would seem difficult to account for such wide variations in power costs as can be figured. The fuel consumption of the better grade Diesel engines is a nearly fixed quantity. Lubricating oil consumption should not vary appreciably. Sufficient records of Diesel upkeep and maintenance costs are now available to allow us to form a sufficiently reliable estimate as to these items. Operating cost, that is, for supervision and labor, however, is of course an item which may vary considerably usually in inverse proportion to the rated horsepower of the installation. But the biggest single variable item is found in the depreciation factor. The annual charge off for depreciation is of course practically unchanged by the load factor. Whether the engine operates 60 or 6000 hours per year has little or no effect on the depreciation factor which remains constant at about 8 percent of first cost per year. This fact taken in connection with the high first cost of making a Diesel installation, especially if standby units are required, will introduce a very large variable factor into computation of total costs per unit of power developed.

The actual operating fuel cost of a Diesel engine in this vicinity,

with prevailing fuel oil prices, will approximate 0.35 cent per kilowatt hour. This figure is based on fuel oil consumption of 0.4 pound per H.P. hour and fuel oil at 6 cents per gallon.

Depreciation charges as explained above may run anywhere from 0.1 to 2.0 cents per kilowatt hour or even higher, if duplication of units is required and load factor is low.

Interest charges will also vary in direct proportion to the amount of service actually rendered by the machine, and in proportion to the ability of the operator to borrow money at low interest rates.

For the reasons given above, it is impossible to make any general statements as to cost of power developed by these engines. Under favorable conditions, however, total operating costs as low as 0.5 cent per kilowatt hour have been shown, while in other cases the total cost may run to 3 or 4 cents per kilowatt hour. Extreme caution is therefore advised before placing too much confidence in an over-optimistic prospectus.

CONTROLLING CONDITIONS

Reasoning backwards from the above remarks, it is easy to see that the conditions which favor economical operation of Diesel engines are:

1—Maintenance of a high load factor over a long period of time.

2—Cheap fuel and lubricating oil.

3—High credit rating of buyer which will enable them to borrow money for installation at low interest rate.

The above remarks may tend to give the impression that there is very little field for the Diesel engine in the water works installation. This is not the case at all. Many engines operating in water works plants today have been furnishing economical and reliable power for driving pumps for many years, and are continuing to create savings which could be effected by no other type of engine. The writer has in mind particularly two small stationary engines in a Virginia municipality which have been in operation now for about nine years during which time they have averaged about 22 hours per day of operation with negligible upkeep and maintenance costs. Several other cases might be mentioned in Virginia where similar records have been made. But it is again recommended that, before making any such installations, the prospective buyer should avail himself of the services of an able engineer to determine first whether Diesel will be suitable and economical and second on the type and proper size of

engine for the job. The selection of proper and economical sized units for Diesel power, especially where variable leading and provision of standby equipment to take care of possible breakdowns is required, is an involved problem, which deserves to be treated with extreme care.

In view of the ever increasing efficiency of these units, it is safe to say that the next decade will see a continued increase in the amount of Diesel power used in this field. Therefore, if you think your power costs are running too high, or if for some other reason, you think a change in type of power required should be considered, serious consideration should be given to Diesel power.

But as a final reminder, when you are giving consideration to Diesel power, be sure, first that the expected economies will be effected; second, that continuity of operation will be assured; and third, that the type is correct.

DISCUSSION

V. VON GEMMINGEN (Town Manager, Culpepper, Va.): Water pumping with Diesel engines has been tried in many instances and where such installations have been made after a complete study of conditions it has been proven to be successful in every respect. The cost of pumping water with a Diesel engine, whether directly connected to pumps or by electric energy generated by a Diesel engine with other methods of pumping, will vary with the particular make of engine, the particular location of the plant, and the size and make of pumps, whether centrifugal, turbine or rotary. The cost of fuel to be delivered to the engine would have a bearing on the cost of pumping, the make of engine whether two or four cycle, number of revolutions per minute, the amount of B. H. P. required, B. H. P. generated and other features which are peculiar to the particular make of engine used; also the amount of load carried by the engines in use. The Diesel engine in itself is an internal combustion engine. However, in appearance and in all of its features it is totally different. Diesel engines are powered by fuel oil and ignited solely by heat resulting from the compression of air supplied for combustion. The depreciation on all such engines is based on the manufacturer's rated B. H. P. of $1.00 per h.p. per year. The maintenance of such engines is questionable today in municipal plants, since the records in most municipal plants are not authentic and are inaccurate. The cost of main-

tenance and depreciation of a high speed engine from 1000 to 2000 r.p.m., is greater than that of a low speed engine of from 300 to 450 r.p.m.

In taking into consideration the cost of pumping water with Diesel engines it is necessary to make comparisons based on like conditions, that is, the make and horse power of the engine, the make and capacity of the pump used, the quantity of water to be pumped, the lift of water, the head, the pressure on the mains, the distance water has to be pumped, the total friction caused by size of mains and the length of water line. In using a Diesel engine, like all other machinery, a maximum efficiency in delivering power depends upon the mechanical peculiarities of the equipment used for such operations. The same holds good for the type of pump designed for this work. It would be obviously unfair to compare any two or more installations, since there are virtually no two plants in which all conditions are identical.

In 1930, an effort was made by Water Works Engineering to get the consensus of opinion in municipal water works plants on the use of Diesel engine units, as a means of power. The following questions were sent by that publication to the engineers and superintendents of water in the different parts of the country:

"In what way have you found your Diesel engine installation more advantageous than equipment previously used? How do the two compare in operating cost? Do you consider it more dependable? Why?"

E. M. Ratcliff, Superintendent, Fayetteville, Arkansas, writes:

"I find the use of Diesel engines in our water works plant more advantageous in that the cost per gallon of water pumped at our Plant has been materially reduced. In our installation of duplicate units we have a better factor of safety."

O. E. Olson, Superintendent, Lindsburg, Kansas, writes:

"The greatest advantage of Diesel engines over our steam equipment is the economical operation both in fuel consumed and the maintenance cost. The KWH is nearly four times as great from the same amount of fuel oil and the cost of repairs is appreciably less."

J. M. Allmendinger, Superintendent of Water in Portage, Wisconsin, says that economy experienced in the use of Diesel engines for water pumping is an outstanding feature. A Diesel engine plant can be shut down for an indefinite period and within three minutes the

plant can be put into operation at full capacity. The total cost of pumping water to the filtration plant and then repumping it into the mains and reservoir with power derived from Diesel engines is 1.3, while the cost with previous equipment was 7 cents per 1000 gallons.

Lewis D. Burns, Superintendent, Pepperell, Massachusetts, says that the total cost of pumping water against a 284 foot head, was $7.00 to $7.20 for fuel and lubricating cost for each million gallons pumped.

The cost of the former equipment for pumping averaged $32.00 per million gallons. The Diesel outfit of 60 h.p. took the place of the old installation consisting of two-25 h.p. 4 stroke ball ignition engine direct connection.

Printed reports showing the comparative cost (in dollars) of pumping water at Belvidere, Illinois, with steam and Diesel power are as follows:

	JANUARY, 1930 STEAM	JANUARY, 1931 OIL	UNIT DIFFERENCE IN COST	
			Plus	Minus
Fuel	547.90	140.20		
Lubricating oil	20.00	36.61	16.61	
Supplies	0.00	2.00	2.00	
Repairs	14.31	0.00		14.31
Labor	497.00	421.91		75.09
Total cost	1,079.21	600.72		478.49
Total water pumped	20,203,300	16,573,600		
Fuel and lubrication cost per 1000 gallons pumped		0.0281	0.0105	
Total cost per 1000 gallons pumped		0.534	0.0362	
Saving per 1000 gallons			0.0172	
Based on 300,000 gallons per year average savings by pumping with Diesel Power		5,160.00		

In a small city with an average pumpage of less than 500,000 gallons per day against a 200 feet head from a river to a settling basin, the relative pumping costs (in dollars) were, as follows:

Pumping station operation per annum, steam unit	3,650.50	
Fuel (cost)	3,521.38	
Total cost (steam)		7,171.88
Cost of operation with Diesel driven units, fuel and lubrication		960.00

Operating cost would include the following:

Maintenance	240.00	
Operating labor	1,200.00	
Depreciation, interest and insurance...	1,800.00	
Total		3,240.00
Total cost		4,200.00

Difference in cost in operating of steam and operating with Diesel driven units:

Steam operated	7,171.88	
Diesel operated	4,200.00	
Total saving		2,971.88

At Culpepper, Virginia, the town installed a 120 B. H. P.-F. M. 2 cylinder, 2 cycle Diesel engine in 1926. The maximum amount of water pumped per day was approximately 250,000 gallons. The total distance from the pumping station to the standpipe is 6,000 feet with a total lift of 400 feet. The average cost of such pumping amounted to $0.0063 per 1,000 gallons. In arriving at these figures fuel cost used was 0.0526 per gallon delivered; maintenance based on $1.00 per B. H. P. per year; 750 gallons high head and 750 gallons low head pump.

In 1934, the town installed electrical pumps, generating its own current in its municipal plant. The current used in operating the electrical pumps is billed to the Water Department at $0.03 per KWH. The records for the municipal plant show that the pumping of water with electrical pumps can be done at a cost of approximately 7 mills per 1,000 gallons. The engines used to generate the energy are 4 cylinder, 4 cycle engines.

(*Presented before the Virginia Section meeting, November 7, 1935.*)

DESIGN OF EARTH FILL DAMS

By Wayne W. Wyckoff

(*Assistant Civil Engineer, Bureau of Water Works and Supply, Los Angeles, Calif.*)

The design of an earth fill dam is governed by two primary considerations, stability and imperviousness. The dam must be stable as a structure and it must be relatively water-tight.

The cross-sectional dimensions of the dam may be governed by the stability or perviousness of either the material in the foundation or the material in the fill itself.

It is usually not feasible to determine the exact dimensions of the dam until the stripping operations are complete. Uniform conditions may not exist over the entire damsite and the stripping operation may reveal conditions not discovered by the preliminary exploratory work.

Frequently the foundation material of the dam has an appreciably higher percolation factor than will exist in the compacted fill, in which case the required length of the path of percolation through the foundation will determine the base thickness and hence the cross-section of the dam. In any event, if the crest width and side slopes of the dam are the same at the abutments as near the center, the shortest path of percolation will normally be from a point on the abutment at the water surface to a point near the downstream toe. A percolation path from the upstream toe to the downstream toe will have the same loss of head as at the abutment while the distance is considerably greater hence the hydraulic gradient is flatter than at the abutment.

The path of percolation through the abutment can be lengthened by placing a compacted fill blanket on the sides of the canyon in front of the dam and extending out from the dam to a point approximately opposite the upstream toe. (See figures 1 and 2.) The same results may be obtained by using a slope sufficiently flat for the entire downstream face but such a method results in a much greater cost. Where tests show an exceedingly high percolation rate through the foundation material, the path of percolation can be increased by extending the compacted earth blanket on the sides and bottom of the canyon

upstream from the toe of the dam for a sufficient distance to reduce the percolation gradient to any desired value.

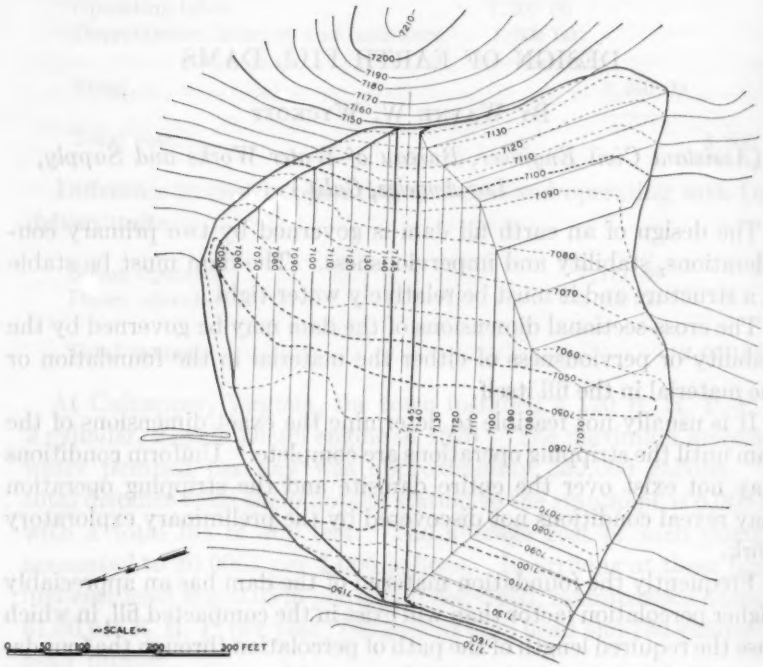


FIG. 1. PLAN OF PROPOSED GRANT LAKE DAM SHOWING EARTH FILL BLANKET ON UPSTREAM SIDE

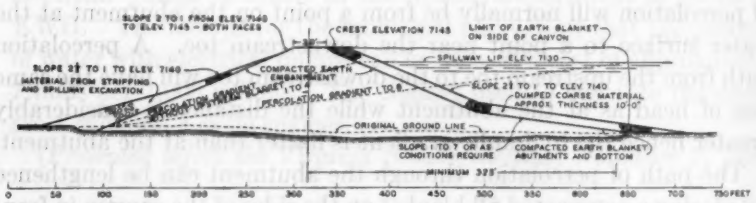

SLOPE 2 TO 1 FROM ELEV 7140 TO ELEV 7145 - BOTH FACES
CREST ELEVATION 7145
LIMIT OF EARTH BLANKET ON SIDE OF CANYON
SPILLWAY LIP ELEV 7130
SLOPE 2½ TO 1 TO ELEV 7140
MATERIAL FROM STRIPPING AND SPILLWAY EXCAVATION
COMPACTED EARTH EMBANKMENT
SLOPE 2½ TO 1 TO ELEV 7140
DUMPED COARSE MATERIAL APPROX. THICKNESS 10'-0"
PERCOLATION GRADIENT 1 TO 8
PERCOLATION GRADIENT 1 TO 8
ORIGINAL GROUND LINE
SLOPE 1 TO 7 OR AS CONDITIONS REQUIRE
COMPACTED EARTH BLANKET ABUTMENTS AND BOTTOM
MINIMUM 325'
0 50 100 150 200 250 300 350 400 450 500 550 600 650 700 750 FEET

FIG. 2. MAXIMUM SECTION OF GRANT LAKE DAM SHOWING PERCOLATION GRADIENTS THROUGH FOUNDATION MATERIAL

If the dam is to be founded on a material which is relatively impervious as compared to the material to be placed in the dam, the percolation rate through the dam itself becomes more important than that through the foundation in which case nothing can be gained by the blankets on the abutment.

RATE OF PERCOLATION

Laboratory tests have shown that most natural soils are of suitable gradation in size of particles such that a method of compaction can be developed which will make them stable and practically impervious to the flow of water. A percolation rate of only a few hundredths of a foot per year under a gradient of unity is quite common. A gradient of unity is the gradient in which the loss of pressure head is one foot for each lineal foot along the path of percolation.

According to Darcy, Hazen, and others, the rate of percolation of water through soils is directly proportional to the percolation gradient. With laboratory tests available for the soils from prospective borrowpits and from the foundation site, the desired length of the path of percolation may be computed such that the rate of flow through the dam or foundation can be held to any desirable minimum. The rate should in all cases be kept sufficiently low to prevent moving of the soil particles. Since the required thickness of the dam as regards percolation through the dam, itself, is usually quite small, the dimensions ordinarily will be determined by the stability of the material under a saturated condition.

In considering stability of the fill, especial attention should be paid the upstream portion. In case the water level in the reservoir is suddenly lowered after the material has reached complete saturation, the direction of percolation will be toward the reservoir. This condition will tend to cause sloughing of the fill material into the reservoir. The face of the dam must be flat enough that the material as compacted will safely resist this tendency to slough. The steepness will be affected by the particular material used and the method of compaction and should be controlled by tests of the saturated plasticity.

For the purpose of illustration of these principles, let us assume that the conditions encountered in the foundation exploration of a damsite indicate that the percolation gradient should be not steeper than one to eight, that is, the length of any possible path of percolation through the foundation is to be eight times the loss of head in order to prevent velocities greater than is desirable. Let us also assume for convenience that the height of the dam above foundation is 85 feet with 15 feet of freeboard and 25 feet crest width. The difference in head from high water in the reservoir to the downstream toe would then be approximately 70 feet. The shortest path of percolation from the reservoir to the downstream toe then should not be less than 560 feet. Assuming an upstream face slope of two and one

half horizontal to one vertical, a downstream slope of approximately five and eight tenths to one would be required to give the desired length of percolation path if no compacted blanket is provided as previously described. A properly compacted fill would be stable on much steeper slopes than five and eight tenths to one, possibly two and one-half to one or steeper and the rate of percolation through the compacted fill would also be so small that such liberal dimensions would not be required. It is apparent then that a compacted blanket extending upstream from this dam will be a more economical method of controlling percolation through the foundation. The thickness of this blanket need not be great since the head tending to produce percolation through it at any point is merely the elevation of the water in the reservoir minus the hydraulic grade elevation of the percolating water immediately beneath the point or simply the amount of the friction loss from the extremity of the blanket to the point in question. These points are illustrated on the dam section of figure 2.

STABILITY OF THE FOUNDATION

In addition to having a sufficiently long path of percolation, the foundation must be sufficiently stable to prevent squeezing out from under due to the weight of the dam. It is obviously not the magnitude of the pressure at any point on the foundation which causes the material to flow but the pressure gradient, that is, the decrease in pressure per unit distance along the foundation. If this gradient can be made sufficiently flat, the dam may rest on practically any type of material. The characteristics of the foundation material at various depths should be determined by laboratory tests before any attempt is made to establish the cross-sectional dimensions of the dam. The plasticity under a saturated condition for the soil must be determined as well as the percolation rate. Field bearing tests of the material in place are of very little value unless the material is saturated at the time of the test. Field and laboratory tests should always be interpreted in terms of conditions imposed by full reservoir.

During the preliminary investigation, exploration pits should be sunk at a sufficient number of places throughout the foundation site to give a fairly complete picture of the conditions to be encountered. Pits are a more valuable means of obtaining information on the foundation conditions than are auger holes since it is possible in pits to measure the dry weight of the material in place at various depths so

that these same dry weights may be duplicated in the laboratory. The results of the field density or dry weight measurements from the pits can be coördinated with logs of auger holes by comparing samples of materials from both pits and auger holes. However, the preliminary explorations are not conclusive and the soil property investigations should be carried on during the stripping and all lenses of undesirable material removed. When it is determined by the plasticity and percolations tests, that the stripping of the foundation has proceeded to such a point that the desired density exists, the design of the dam may be completed in conformity to the principles already

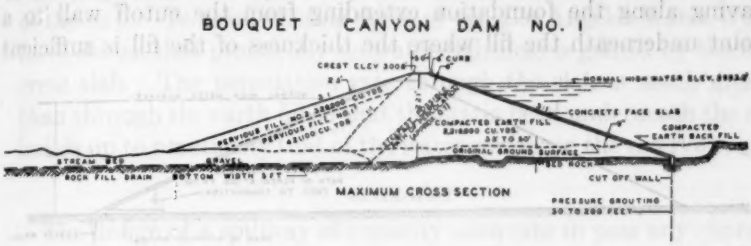


FIG. 3. SHOWING STEEPEST POSSIBLE PATH OF PERCOLATION THROUGH THE DAM OR ABUTMENTS

Earth fill data:

Compacted earth fill

Percolation velocity less than 0.15 ft. per year

Pervious fill no 1

The material was the least pervious material from stripping and rock quarries. The material was graded with the most pervious material being placed near pervious fill no. 2.

Pervious fill no. 2

The material was the most pervious material taken from strippings and rock quarries.

outlined. In case the foundation material is rock, it is practically impossible to determine the extent to which the rock is fractured until the stripping is complete.

CUTOFF WALLS

If the dimensions of the dam are properly determined to limit the percolation gradient, there is usually very little use for a cutoff wall. Unless it is possible to extend the cutoff wall into some formation which is more impervious than that which the dam rests upon, it can accomplish very little other than lengthening the path of percolation a slight amount. This lengthening can be accomplished much more

economically by extending the earth blanket along the abutment as has already been explained. About the only useful purpose such a cutoff wall will serve is as an anchorage for the concrete face slab if such is used.

If the dam is founded on rock, grout holes should be put down in the vicinity of the upstream toe in order to completely seal any cracks which might extend underneath the dam to become possible channels for percolation. If a cutoff wall is used the grout holes should extend down through and beneath the wall.

In this case, it may be advisable to place a well reinforced concrete paving along the foundation extending from the cutoff wall to a point underneath the fill where the thickness of the fill is sufficient

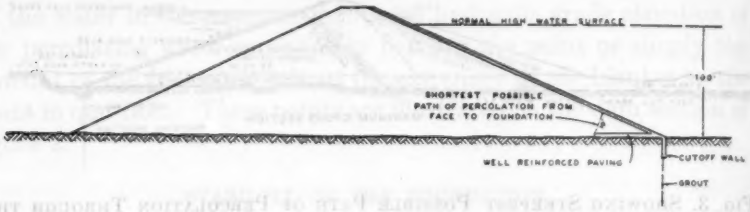


FIG. 4. TYPICAL SECTION OF EARTH FILL DAM ON ROCK FOUNDATION
Illustrating a possible method of treatment near upstream toe

Assumptions:

Allowable rate of percolation through the dam, 2 feet per year

Fill to be compacted so that percolation rate is 0.2 feet per year, with grade of unity.

Minimum allowable gradient = 10, or shortest path of percolation should be one tenth the head loss along the path.

Foundation rock impervious if all fissures are grouted.

to prevent excessive downward percolation to any possible fracture in the foundation. (See figure 4.)

This procedure is based upon the assumption that the foundation rock is impervious except where fissures occur. The grouting and the paving on the foundation prevent the possibility of a short path of percolation into the fissures under the dam from points in the reservoir near the toe.

FACE PROTECTION

Unless the upstream face of the dam is protected, considerable cutting may result from wave action. This protection may consist of coarse rock riprap, a concrete slab or other material, the nature of which may be determined by economic considerations.

If the rock riprap is used, it is desirable to place the finer material

next to the earth fill to prevent the erosive action as each wave recedes.

If a concrete slab is used, the slab should be provided with either sufficient reinforcing steel to prevent large shrinkage cracks, or with expansion joints in which suitable water stops are inserted. Cracks larger than $\frac{1}{8}$-inch have been found to permit the flow of a sufficient quantity of water to cause gradual undercutting of the slab. The slab may be anchored at the abutment to a concrete wall so designed to anchor against shrinkage and temperature stresses, and of sufficient depth to eliminate the possibility of undercutting at the edge of the slab. Tests on existing dams of the Los Angeles Water Works have shown that practically no water-tightness is gained by the concrete slab. The percolation rate through the slab is much greater than through the earth fill so that the static head underneath the slab builds up to practically that of the water surface in the reservoir.

SPILLWAY

The design of a spillway of capacity adequate to pass any conceivable flood is of paramount importance. The estimate of flood flows is a problem in itself to be solved according to the particular location and characteristics of the watershed. However, this particular problem is not considered to be within the scope of this discussion.

OUTLET WORKS

The design of the outlet works is a problem depending upon the particular service the reservoir is to render in the water system. If the outlet pipe passes through or under the dam, care should be exercised in determining its location. The exact location cannot usually be selected until the stripping of the dam site is practically completed. If appreciable settlement of the foundation occurs due to the superimposed weight of the dam severe stresses will occur in the pipe or even complete failure may occur. If the conditions are fairly uniform throughout the entire foundation, the settlement will be so distributed that severe stresses will not occur. In certain cases it may be desirable to locate the outlet in a tunnel through the abutment of the dam.

SUMMARY

If a dam is designed in accordance with the principles set forth herein and based upon careful laboratory and field tests of the foundation and available borrowpit material, the structure is certain to be safe and sufficiently impervious to leakage.

(*Presented before the California Section meeting, October 25, 1935.*)

SOIL COMPACTION CONTROL FOR ROLLED EARTH DAM CONSTRUCTION

By R. R. Proctor

(*Bureau of Water Works and Supply, Los Angeles, Calif.*)

Many engineers consider soils for earth dam construction on the basis of water-tightness alone, without regard to the extent to which the soil should be compacted to resist softening from saturation. Attempts have been made to compute the position of the "Line of Saturation" in an earthfill dam, in order to predict the margin of safety, taking into consideration the weight of that part of the dam lying above the "Line of Saturation," supposedly not saturated, thus lending an undetermined amount of support to the saturated portion of the dam;—only too often the majority of the cross-section.

The possibility of error by use of this method was very forcefully brought to the attention of the writer about five years ago, while investigating the suitability of a soil for use in an earth dam.

The usual screen analyses had been secured for this soil, according to the then existing practice, and the results proved it to be very suitable for use in a dam. Accordingly, a percolation cylinder similar to figure 1 was filled with soil to a depth of 12 inches. The soil was compacted by an arrangement of levers designed to exert the same pressure on the soil, by means of a sheepsfoot tooth taken from a roller, as would be exerted on the soil by the tooth when in use on the roller. The soil was thus compacted to a very hard and firm condition, appearing to be a suitable earthfill for use in a dam.

The two gallon water bottle was then filled with water and percolation started. The writer returned on the following day to observe the quantity of water that had passed through the specimen and, to his surprise, found the two gallon bottle empty and the soil in the cylinder to be very wet and soft; in fact, it was possible to push the hand entirely into the soil from the top! It was apparent from this experiment that an entirely different system of compaction control was absolutely necessary, if any known degree of safety was to be accomplished from compaction of this soil in a dam! It was also

apparent that the saturated plasticity of the compacted soil was much more important than water-tightness alone.

Twelve percolation cylinders were then constructed, similar to the one shown, and some of the same soil was compacted into each of the cylinders by the same method previously used, except that the moisture content was varied from high values in the first cylinder to low values in the last. The specimens that were compacted at the highest and lowest moisture contents were the most pervious; the dry com-

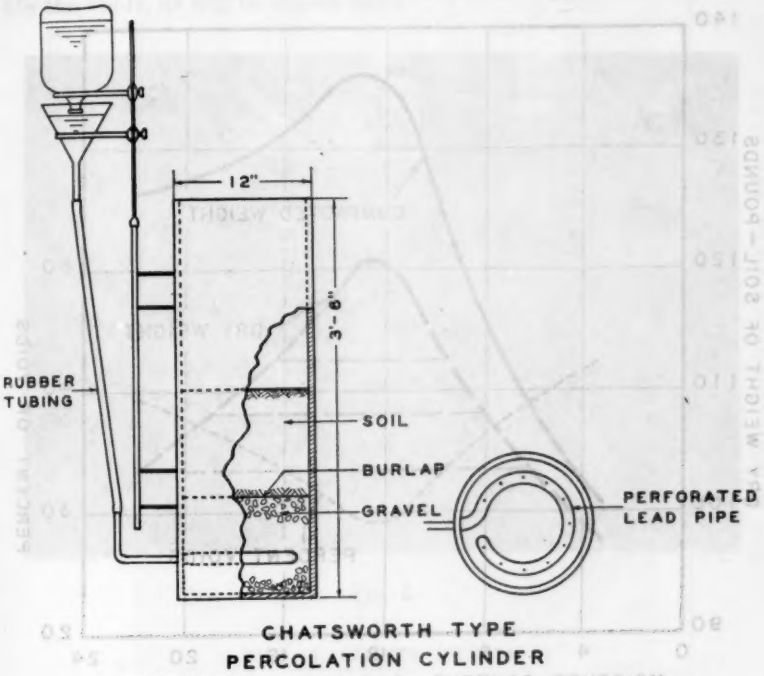


FIG. 1

pacted specimens became saturated and similar in appearance and water-tightness to those that were wet when compacted. Two of the specimens which were compacted at intermediate moisture contents were, as closely as could be measured with this device, water-tight. The surface of the specimens dried, indicating a percolation rate less than about 2 feet per year.

It was apparent from these tests that this particular soil was most suitable for use in a dam when it was compacted at an intermediate

moisture content, although this was more moisture than had been previously considered correct. The difficulty with the original specimen was that it had been compacted too dry and, while an excellent appearing fill, it was actually unsafe for use in a dam in that condition.

Figure 2 presents the results of experimental compactions of a particular soil, at a wide range of moisture content. The weight of the soil per cubic foot after compaction is shown at the left. The

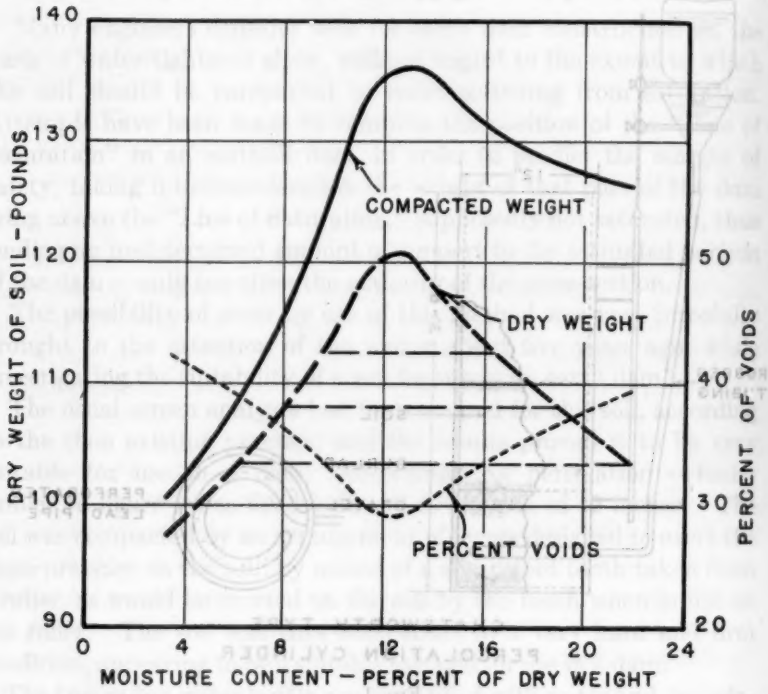


FIG. 2

moisture content of the soil is shown at the bottom and the percent of voids, or porosity, is shown at the right. It will be seen that, as the moisture content of the soil was increased, the compacted weight increased until a maximum was reached, after which the addition of further moisture to the soil reduced its compacted weight.

The lower curve shows the actual dry weight of the soil, obtained by deducting the weight of the water after compaction. The percent of voids curve shows a decrease and an increase in voids corresponding

to the increase and decrease in dry weight. This curve supplies an explanation of the results secured from original experiment with the 12 samples. The percolation cylinders which were compacted at intermediate moistures and did not soften and were practically watertight must have been compacted under conditions similar to these at the peak of the dry weight curve. Compaction is limited by moistures greater than that at the peak of the dry weight curve. The soil under these conditions is compacted until the water practically fills the voids, as will be shown later.

FIG. 3

THE PLASTICITY NEEDLE

The twelve preliminary percolation tests indicated a need for an instrument to measure the plasticity of the compacted soil. The instrument shown in figure 3 was constructed to measure the resistance of soil to penetration at a rate of about ½-inch per second. The pressure required to effect this penetration is expressed in pounds per square inch of penetrating area. Several additional uses were found for this instrument. It is used to measure the plasticity of the experimentally compacted specimens, and, with such results available for a particular soil, the moisture content of the soil can be found by compacting it in a container, measuring the penetration resistance,

and referring to the penetration resistance curves that had been previously prepared. This probably is the most important single use of the needle as it enables the quick determination of soil moisture content in the field, without delay caused by the slowness of oven drying. The entire test requires about two minutes.

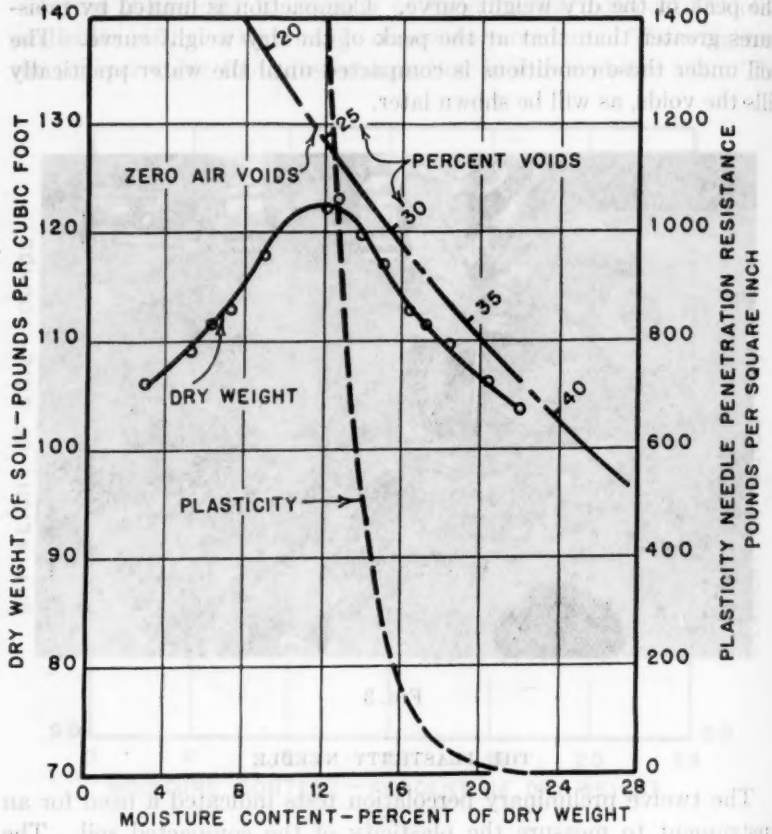


FIG. 4

The needle is also used to control the moistening of soils in borrowpits; the preliminary compaction and plasticity curves indicate the maximum moisture content and plasticity that should be permitted during compaction. A soil in the borrowpit can be compacted in the cylinder and the needle reading taken to determine this. The needle can also be used to determine the plasticity of any natural soils in place that are known to be saturated.

Figure 4 shows the results of compaction—plasticity tests on a particular soil. These curves can be used to predict the ultimate softening, from saturation, that may be expected when this soil is compacted or found in place at a known dry weight. It may be assumed that the soil was found in place at 8 percent moisture and a dry weight of 110 pounds per cubic foot. The zero air voids curve

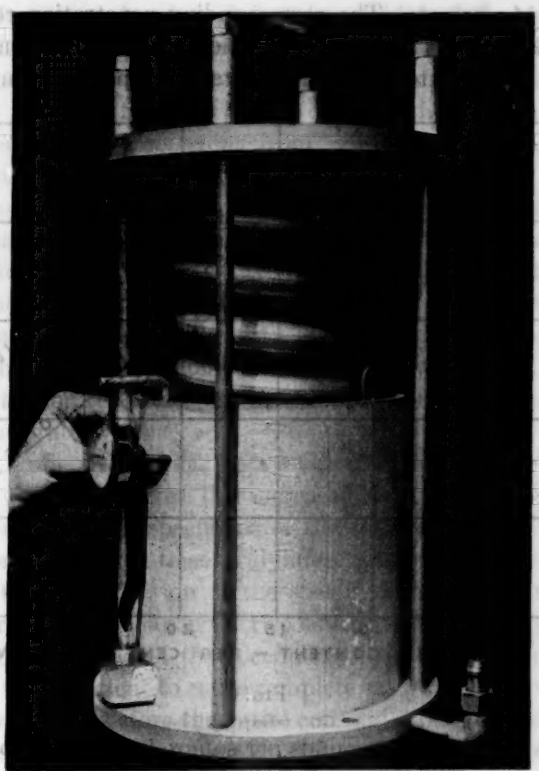


FIG. 5

indicates that, when fully saturated, the moisture content of the soil will be 20 percent. The plasticity curve indicates that this soil, when compacted at 20 percent moisture, has a plasticity needle penetration resistance of 20 pounds per square inch, which is too low a value for safety in a dam or foundation. A saturated penetration resistance of about 300 pounds per square inch is desirable for most

dams. The compaction-plasticity curves can be used to determine the minimum dry weight and compacted plasticity required to accomplish this. The plasticity curves indicate that a maximum moisture content of 15 percent is necessary for a penetration resistance of 300 pounds per square inch. The soil will have this moisture content when saturated at a dry weight of 120 pounds per cubic foot. The maximum moisture content that permits compaction to this dry weight is 14 percent. The corresponding penetration resistance is 500 pounds per square inch. It is evident that this soil must not be moistened to the extent that its resistance to penetration after com-

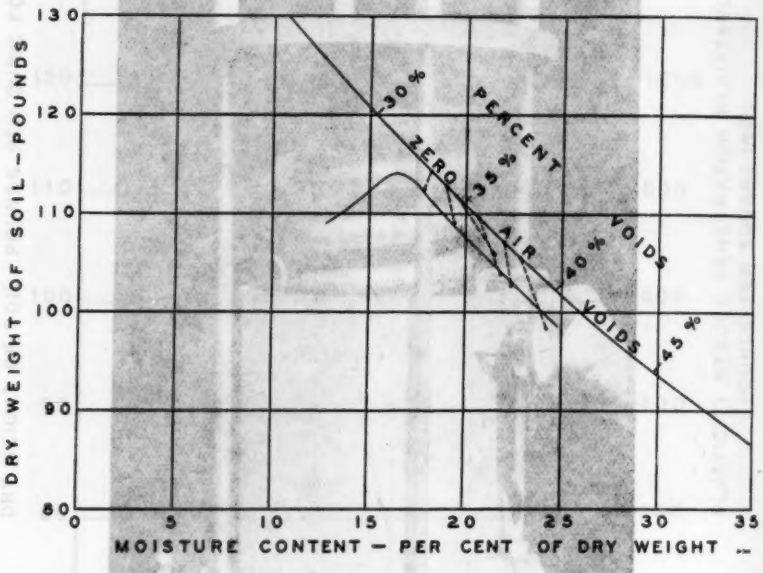


FIG. 6

paction is less than 500 pounds per square inch. The lower limit of moisture content is that which results in a compacted penetration resistance of about 2500 pounds per square inch for the heavier rollers.

The percolation-consolidation cylinder now being used in place of the crude device with which the original experiments were performed is shown in figure 5. This device can apply a load on the soil equivalent to that which may exist at any part of the dam from the weight of the fill above. The amount of consolidation that takes place can be measured with a strain gauge as shown and the percolation rate is determined by measuring the quantity of water that

enters the specimen. A 2 to 3 inch soil layer is used, water is applied to the bottom of the layer at a pressure head of about 12 feet and a loading equivalent to 10 feet of fill depth is usually applied to the top of the soil specimen.

The consolidation that takes place under a 10 foot depth of fill load is about all that should be considered, further consolidation should not be relied upon to secure the necessary dry weight or soil density to resist softening, unless it is definitely established that water may flow from the soil with sufficient rapidity to permit the consolidation to take place within the required time. It is a much better practice to compact the soils to the required density, thus eliminating any possibility of excessive softening from saturation before the settlement or consolidation has taken place.

The percolation rates of practically all soils, compacted to provide sufficient stability when saturated, have been found to vary from a few thousandths of a foot per year to not much over one foot per year; however, the majority of soils have a percolation rate less than 0.1 foot per year—a negligible quantity. This percolation rate applies to soil specimens that were compacted sufficiently to have a plasticity needle penetration resistance of not less than 300 pounds per square inch when saturated.

The results of consolidation from 6 Grant Lake samples are shown in figure 6 which indicates the increased dry weights secured by consolidation of several specimens of the same soil under a 10 foot fill load after compaction to several different dry weights. It will be noted that the consolidation of the specimens at lower dry weight was considerably more than in the two having the highest dry weights. The consolidation in all of the specimens, except the one of highest dry weight, was sufficient to cause complete saturation and water was forced out from four before the entire consolidation could take place.

This discussion of soil properties is by no means complete, it is only an outline of the fundamentals of the soil compaction control that has been developed and used by the Bureau of Water Works and Supply of the City of Los Angeles during the past few years.

(*Presented before the California Section meeting, October 25, 1935.*)

LIST OF SECTION PAPERS

Second Joint Annual Conference of the Virginia Section of the A. W. W. A.

AND

Virginia Sewage Works Association

Roanoke, Va., November 7–8, 1935

Economies in the Selection and Operation of Electric Power for Pumping Equipment ..... D. P. Minichan
The Diesel Engine in the Waterworks Field ..... W. S. Russel
The Lubrication of Water Works Equipment ..... W. B. Phillips
The Painting of Steel Tanks and Fire Hydrants ..... C. F. Harris
The Railroad's Problem of Water Supply ..... F. P. Turner
Changes in Waterworks Practice and Their Influence on Fire Service ..... A. C. Hutson
Investigation of Water Supplies in the Shenandoah National Park ..... J. J. Dirzulaitis
Some Practical Aspects of Handling Chlorine ..... Chris F. Bingham
Williamsburg's New Sewage Disposal Plant ..... A. L. Meisel
Lynchburg's New Transmission Main ..... Richard F. Wagner
Transite Pipe ..... Motion picture by C. A. McGinnis
Waterworks and Sewerage Improvements under P.W.A. ..... James A. Anderson
Developments and Trends in Modern Water Works Practices ..... L. H. Enslow
Operating Kinks for Filter Plants ..... F. E. Stuart
Round Table and Question Box

ABSTRACTS OF WATER WORKS LITERATURE

FRANK HANNAN

Key: American Journal of Public Health, 12: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the journal.

Long Life for Wood-Stave Pipe Indicated by Denver Experience. Eng. News-Rec., 114: 130, January 24, 1935. Condition of existing wood-stave pipe lines in Denver water system and maintenance and replacement policies of Board of Water Commissioners are described briefly. At one time, Denver had more than 100 miles of wood-stave conduits in use: at present time, mileage is 73. A number of years ago, policy was adopted of suspending further construction of wood-stave lines for filtered water, but not for raw water; chiefly because of unfavorable terrain. Major portion of filtered water now reaches city through 60- and 66-inch conduits of concrete and steel. In wood-stave lines, after some years, breaks, especially at summits, are a source of trouble. After experimentation, perforation of staves, to ensure their saturation, as suggested by FRED BINNER, has been adopted as means of combating dry rot and thus prolonging life of lines. Perforations, $\frac{3}{8}$ inch in diameter, are drilled from inside to within less than $\frac{1}{4}$ inch of outside of staves. Holes are arranged in staggered groups of 3 (across the nearly 6-inch width of stave) spaced at 3-foot intervals. Three holes were found necessary in order to ensure saturation across width of stave. Saturation is maintained by above procedure under pressure as low as 15 pounds. As added precaution against decay, outer surface of staves, before assembly, is coated with carbolineum, and outside of completed line is coated with asphalt paint. Continuous wood-stave pipe was invented by C. P. ALLEN, former chief engineer of Citizens Water Company and later of Denver Union Water Company. First pipe of this kind was laid in 1884, at point which is now within city limits of Denver.—*R. E. Thompson.*

A Decade of Development in the Oxy-Acetylene Processes. G. O. CARTER. Eng. News-Record, 114: 249, February 14, 1935. Advances in welding and cutting with oxy-acetylene are reviewed. To-day, at least 3 times as much metal is deposited with same amount of gas in flame as was done in 1924.—*R. E. Thompson.*

California Supreme Court Establishes New Water Law. Eng. News-Rec., 114: 231, February 7, 1935. Establishing rule of "reasonable use" and repudiating 50-year old theory of riparian rights, under which riparian owners of California demand undiminished flood flow of streams, regardless of use, California state supreme court on January 31 announced unanimous decision in

case of Peabody *vs.* City of Vallejo. This test case will establish new precedent in western water law. Under decision, use of all waters of state, riparian or otherwise, will be governed by doctrine of reasonable use, and flood flows, which under riparian rights doctrine were wasted into ocean, may now be appropriated and applied to beneficial use.—*R. E. Thompson.*

Fresh-Water Reservoir Built for Aquarium. R. LEEDOM. Eng. News-Rec., **114:** 158–9, January 31, 1935. Brief illustrated description of concrete reservoir built as relief project for New York Aquarium. Reservoir, which has capacity of 50,000 gallons, will be used to store clear water obtained during summer months from city supply for use during year. Special precautions were taken in waterproofing structure to prevent contamination by sea or surface water and in order to avoid deleterious action attending contact of water with metal of any kind, hard rubber piping and fiber pumps were employed. A filter was incorporated for purifying water returned from specimen tanks. Sea water, with minor additions, has been filtered, purified, and re-used for over 25 years with great success for salt water specimens.—*R. E. Thompson.*

Reading Water Tunnel, Dam, and Conduit. Eng. News-Rec., **114:** 66, January 10, 1935. Unit prices are given from bids received on following contracts for Reading's new Maden Creek water supply system: (1) half-mile concrete-lined tunnel of 6-foot-9-inch finished bore; (2) earth dam (Ontelaunee); (3) ¾-mile precast concrete pipeline.—*R. E. Thompson.*

New Chemical Building for Toronto's Water Supply. Eng. Cont. Rec., **49:** 83–4, February 6, 1935. Brief description of new chemical building erected at John Street Pumping Station, Toronto, in 1934 at approximate cost of $75,000, to house equipment for dechlorinating water supply. Equipment consists of 7 sulphonators (similar to vacuum pedestal type chlorinators) having combined capacity of 2100 pounds per day, 2 evaporators, and 2 indicating and recording 6000-pound scales. One-ton sulphur dioxide containers are delivered directly into elevator on ground floor, from which they are unloaded on floor of evaporating room by means of mono-rail system which also enables them to be placed in position in any part of room. Both operating and evaporating rooms are equipped with high capacity fans to exhaust any gas inadvertently discharged. Manifold carrying sulphur dioxide from evaporators to machines consists of streamline copper tubing and discharge lines from machines to suction well are made up of air drill hose and Inconel metal tubing. Ordinary corrugated rubber hose was found unsatisfactory for carrying sulphurous acid solution, but air drill hose has given excellent service. It is hoped that the Inconel tubing will obviate necessity for periodic renewal of rubber hose lines.—*R. E. Thompson.*

Winter Maintenance of Water Supply Systems. Eng. Cont. Rec., **49:** 84–5, February 6, 1935. Instructions issued by Associated Factory Mutual Fire Insurance Companies regarding thawing of water mains and maintenance of hydrants and indicator posts in cold weather are given. Frozen mains can usually be thawed quickly by electric current of 300 to 1300 amperes, voltage

of 50 to 100 being ordinarily sufficient. Compound joints may render electric thawing impracticable. To reduce fire hazard from current diverted into parallel circuits, it is usually advisable to disconnect ground wires in adjoining buildings, and to take other precautions as determined by experienced electrician. Use of steam, by inserting steam hose or flexible copper tubing through hole drilled in frozen section of main is also effective. Fire hydrants may be thawed electrically, or by steam. Alcohol poured into hydrant barrel may soften ice sufficiently to permit removal by flushing. Use of calcium or sodium chloride is not recommended owing to corrosive action. Use of Thermit is not advisable owing to difficulties involved and fact that it generates comparatively small quantities of heat at dangerously high temperatures.—*R. E. Thompson.*

Progress on Large Projects (Water Supply). Eng. News-Rec., 114: 201-2, February 7, 1935. **El Capitan Dam.** Construction of $3,000,000 El Capitan Dam for city of San Diego was practically completed during year, and runoff from San Diego River is now being stored in the 118,000-acre-foot reservoir. Dam, which is 220 feet high, is combination rock and earthfill structure. Future work will include pipe line from dam to existing conduit some 8 miles downstream, and water treatment plant. **Colorado River Aqueduct.** Tunnel driving has been major feature; at end of year 56 of the 91 miles of tunnel had been driven and concreting started at several points. Work has been started on 145 miles of canals, conduits, and siphons, and in distributing system contracts have been let and early start is to be made on Monrovia and Pasadena tunnels, which total 10 miles in length. Transmission line that is to furnish power for the 5 pumping plants along aqueduct will be under construction this year. **Parker Dam.** At intake of Colorado River Aqueduct, Parker Dam is being constructed across Colorado River from California to Arizona shore at point where foundation conditions necessitate dam 320 feet high to raise water surface 85 feet. Dam will be concrete structure, arched in plan and 800 feet long on crest. Bureau of Reclamation undertook construction of dam for Metropolitan Water District of Southern California and awarded contract upon which preliminary construction started in September. Subsequently, work on dam was stopped on account of objections by Arizona. Contractor is carrying forward such works as can be done on California side until dispute is settled. **Rodriquez Dam, Mexico.** Good progress during year on Rodriquez Dam, south of San Diego, brings near to completion this buttress-type structure of record height. **Madden Reservoir, Canal Zone.** Dam forming Madden Reservoir to supplement water supply of Panama Canal was completed late in year. Storage of water had been begun previously, and water was discharging over spillway before end of December. **All-American Canal.** Thirty miles of ultimate 80-mile main canal is under construction. **Moffat Tunnel Water Diversion.** Denver Municipal Water Commission has perfected arrangements for $3,500,000 PWA loan for enlarging and lining pioneer bore of Moffat Tunnel to divert Fraser River water under continental divide into the eastern slope watershed. Bids have been received and award made pending only approval by PWA. **St. Louis Waterworks.** Completed section of 13-mile pipe line from Stacy Park Reservoir to St. Louis totaled

19,000 feet at end of year. Line will consist of 47,000 feet of 60-inch, 11,000 feet of 54-inch, and 10,000 feet of 48-inch steel pipe, delivered in 40-foot lengths. **Chicago Ave. Water Supply Tunnel.** Work on 12-mile tunnel and new Dever crib, together costing $13,000,000, progressed at uniform rate during 1934. Crib is practically completed, and tunnel will be ready for use by July 1. New Cermac electrically driven pumping station (formerly Harrison St.) will replace 2 obsolete steam-operated units and one motor-driven unit. With necessary tunnel connections, station will cost $2,250,000. Foundations and tunnel connections are complete, and work on superstructure is progressing. Operation is set for July 1. **Milwaukee Purification Plant.** Cofferdam around site of proposed 200-m.g.d. filtration plant offshore in Lake Michigan near North Point pumping station at Milwaukee is ready to be pumped out so that concrete substructure can be placed. A 750,000-gallon underground concrete wash water tank has been constructed near shore in public park on high bluff. Contracts to date amount to $300,000. **New York City Water Supply.** Contracts totaling $764,000 were awarded during year for valves, gates, etc., required for operation of second delivery tunnel, known as city tunnel No. 2, extending from Hill View Reservoir in Yonkers through boroughs of Bronx and Queens and into Brooklyn. Entire tunnel has been completed except for installation of operating equipment, testing and unwatering. Expected that tunnel will be placed in service during latter part of 1935. **Boston Water Supply.** Progress on Quabbin Reservoir project of Metropolitan Water Supply District of Boston during 1934 included completion of 1500-foot pneumatic-caisson cutoff wall at Quabbin Dike, completion of last 10-mile section of 24.6-mile Wachusett-Quabbin Aqueduct (except for finishing 2 shafts), recent award of contract for 2,500,000 cubic yard Quabbin Dike, to be built by full hydraulic method, and miscellaneous construction operations. To remove sewage from several institutions located on watershed, 9-mile line of 10- to 16-inch cast iron pipe was completed, tying in with sewerage system of Worcester. Relocation of highways in reservoir area was continued, 21 miles of 30-foot bituminous-surfaced roads being built during year on west side. Highway relocation on east side will be carried on during coming year. Bids are soon to be called on cutoff wall of main dam, to be built on Swift River to create reservoir in conjunction with Quabbin Dike. Exploratory caissons have been sunk at site.—*R. E. Thompson.*

Maintaining Vancouver's Submerged Pipe Lines. W. H. Powell. Eng. News-Rec., 114: 124-5, January 24, 1935. Since 1889, Vancouver, B.C., has at various times placed submerged pipe lines across Burrard Inlet at First and Second Narrows. In July, 1931, there were in operation two 12-inch and four 18-inch pipes across First Narrows and ten 18-inch pipes across Second Narrows. In general, lines are 1800 to 2500 feet long. In July, 1931, contract for First Narrows pressure tunnel was awarded, and on completion in June, 1933, pipes in that area were abandoned. Breaks had been continually occurring due to mains being struck by ships or ships' anchors, settlement, etc. Brief details are given of maintenance and repair operations and removal of abandoned lines. Practically all the lead from joints was salvaged and of pipe removed, 95 percent can be reconditioned and reused. Interior of recovered

pipe was badly tuberculated. On exterior, where pipes had been wholly covered or wholly bare there was little indication of wear, but where pipes had been partly covered, effect of moving sand and gravel, especially if there had been any leakage, was quite apparent.—*R. E. Thompson.*

Boulder Dam Storage Is Begun; Colorado River Under Control. Eng. News-Rec., 114: 232, February 7, 1935. On February 1, about 18 months ahead of original schedule, Colorado River was put under control and storage in Boulder Dam reservoir started with closing of bulkhead gate at upstream portal of diversion tunnel No. 1. Revised plan of closure, worked out last fall, is outlined briefly.—*R. E. Thompson.*

Hydrological Data from the Miami Conservancy District. C. S. BENNETT. Eng. News-Rec., 113: 556–8, November 1, 1934. Data pertaining to rainfall, runoff, etc., accumulated by District are presented, expanding previously published information.—*R. E. Thompson.*

Drought, Weather, and Streamflow. JOHN C. HOYT. Eng. News-Rec., 114: 199–200, February 7, 1935. Severe drought conditions over large area of country characterized year 1934. Drought was not result wholly of shortage of total precipitation, but rather of unequal distribution of precipitation throughout year, together with excessive heat and warm winds. Worst effects were confined to semi-arid and arid areas. In most sections, excess precipitation during fall was especially favorable for replenishment of depleted surface and ground water supplies, and conditions were generally favorable for 1935 to be good water year. As result of experience during drought of 1930–1, there had been, throughout country as whole, material extension of facilities for obtaining water, and consequently effects of drought of 1934 on water for domestic and industrial uses were greatly lessened. We have reached time in development of country when, if serious consequences are to be averted, much more careful consideration must be given to water supply than in past. As national development increases, detrimental effects of droughts increase, and they will continue to increase unless proper relationship between water supplies and requirements is established and maintained. Effects of droughts of 1930–1 and 1934 emphasize need for more complete and systematic studies on continuing basis of country's water resources and of possibilities for their development.—*R. E. Thompson.*

Preparing New Maps of the Tennessee River Basin. T. P. PENDLETON. Eng. News-Rec., 114: 243–4, February 14, 1935. United States Geological Survey, in coöperation with Tennessee Valley Authority is engaged in mapping basin on uniform scale much larger than existing maps. Scale of 1 inch = 2000 feet was adopted. This is larger than scales usually employed for maps of large areas, yet not so large that completion will be materially retarded. To cover area on this scale, 766 maps will be made. Topographic map is more desirable than maps of other types, but cost and time required made it advisable to defer consideration of this type. Map of planimetric type, which shows all features and information appearing on topographic map except con-

tour lines, was selected as best substitute. In event that topographic mapping is needed in special areas, contour lines will be drawn on planimetric map as base. Aerial photography was employed, using camera of multi-lens type, field of view embracing about 140°. At altitude sufficient to yield scale of 1 inch = 2000 feet, camera records view some 12 miles wide and 84 square miles in area. Photography of this area of 40,600 square miles was accomplished in 8 months. Work involved exposure and development of 50,000 negatives. Procedure is outlined—*R. E. Thompson.*

Prestressed Concrete Pipe Tested at Purdue. RAY B. CREPPS. Eng. News-Rec., 114: 275, February 21, 1935. Comprehensive studies have been made at Purdue University on 30-, 42-, and 60-inch diameter prestressed concrete pipes, with exact measurements of stresses in steel rods and concrete when pipes were being prestressed and with internal fluid pressures in the prestressed pipes. Bond between steel and concrete was prevented by coating rods, and both hoop and longitudinal rods were provided with threaded ends and nuts in castings in wall of pipe so that they could be prestressed in tension, in order to induce compression in concrete. Results are shown graphically and discussed briefly.—*R. E. Thompson.*

Manganese and Iron Removal Plant for Lincoln, Nebraska. Eng. News-Rec., 114: 246–7, February 14, 1935. Manganese in objectionable amounts (>0.1 p.p.m.) in new Platte Valley well water supply of Lincoln, which was constructed at cost of $2,300,000, made its presence known suddenly, 14 months after supply had been turned into mains. Overnight, tap water became turbid, having appearance of diluted black ink. Investigation revealed 6-inch sludge deposit in reservoirs and gelatinous coating in 25-mile, 36-inch pipe line, brownish red at Ashland end and black at Lincoln end. Old wells within city provided sufficient capacity for immediate needs, and after cleaning of reservoirs and flushing of pipe line it has been found possible to use new supply if systematic flushing is carried out every 3 weeks. It was found that manganese content of the water of the 5 wells varied from 0.25 to 1.25 p.p.m. and iron content from 0.3 to 0.6 p.p.m. Change in ground water level is considered possible contributing cause. Experimental plant was installed providing for tray aëration, sedimentation, contact filtration, and rapid sand filtration, from operation of which following conclusions were drawn:—(1) rates of aëration as high as 40 gallons per square foot per minute effected satisfactory carbon dioxide removal and as high as 90 per cent saturation of oxygen; (2) contact filters containing pyrolusite (manganese ore) gave satisfactory manganese removal at first, but after 30 days efficiency began to decrease rapidly and at end of 60 days, filters were completely ineffective: particles of pyrolusite apparently became coated with brownish gelatinous material that seemed to interfere with process: it was found possible to clean pyrolusite and restore filter to original efficiency by washing with 5 percent solution of sulphuric or hydrochloric acid; (3) application of 1 p.p.m. chlorine immediately after aëration restored efficiency of pyrolusite, but this was effective only for few days: later, it was found that satisfactory results could be secured by chlorinating ahead of contact filters containing gravel or coke; (4) coke in

contact filters gave practically same results as gravel; (5) there was no apparent benefit from sedimentation basin, when prechlorination was employed; (6) filtering rate of 4 gallons per square foot per minute in final sand filters was entirely satisfactory; (7) satisfactory iron and manganese removal could be secured by addition of 2.5 grains lime per gallon followed by 2 hours' sedimentation and final sand filtration; and (8) satisfactory iron and manganese removal could be obtained by:—(*a*) aeration, (*b*) chlorination (0.5–1.0 p.p.m.), (*c*) upward flow contact filters containing coke, gravel, or pyrolusite, (*d*) 2-hour sedimentation, and (*e*) rapid sand filtration. These processes have been incorporated in design of new plant. While experimental plant results indicated that better results were possible without sedimentation, 2-hour basin is to be provided for future water softening and as additional safeguard for present operation. Aërator consists of 3 units of 4 trays each. Tray bottoms are of slate, with 1¼-inch holes spaced on 6-inch centers. Four inches of coke will be placed on each tray. Aërator is designed to operate at 28 gallons per square foot per minute at 12-m.g.d. rate and 42 gallons per square foot per minute at 18-m.g.d. rate. The 2 contact filters are to be of upward flow type, operated at 8 gallons per square foot per minute. Design of these filters permits conversion to standard rapid sand filters if softening is adopted. Filter media will be pyrolusite, coke, or gravel, depending on outcome of experimental plant operation. Sedimentation basin will have capacity of 1 million gallons and final filters, of which there will be 4, of 3-m.g.d. capacity, operated at 4 gallons per square foot per minute, will be of rapid sand type. Clear well of 100,000-gallon capacity will be located below 2 of filters. Two electrically-driven centrifugal pumps of 7.5 and 11 m.g.d. capacity will pump water from clear well to existing 3-million gallon reservoir. Contracts for entire plant have been awarded, totaling about $211,000 and plant is under construction, completion being scheduled for May 15, 1935.—*R. E. Thompson.*

Field-Testing Devices for Hydraulic Fills. E. A. Rowe. Eng. News-Rec., 114: 150–2, January 31, 1935. In construction of El Capitan Dam, combination rock and hydraulic-fill structure for City of San Diego on San Diego River, city maintained complete laboratory for physical tests of materials, etc., but contractor found that work could be expedited if determinations could be made as work was performed. Accordingly, author devised simple equipment for core sampling, fairly accurate determinations of moisture content and voids, and screen analysis. This equipment is described and illustrated.—*R. E. Thompson.*

Treating Water With The Aid of Magnesium Metal. Hermann Bach. Water Works and Sewerage, 81: 7, 241–243, July, 1934. Magnesium metal reacts with chlorine dissolved in water in two ways simultaneously: (1) $Mg + 2Cl = MgCl_2$; and (2) $Mg + 2Cl + 2H_2O = Mg(OCl)_2 + 2H_2$, forming magnesium hypochlorite (analogous to calcium hypochlorite). The hypochlorite chlorine is only loosely bound and upon slight acidification (CO_2 content of air) is set free: (3) $Mg(OCl)_2 + 2CO_2 = Mg(CO_3)_2 + 2Cl$. Furtunately, when period of contact is short, reaction (1), forming magnesium chloride, will predominate; but some hypochlorite will also be formed. By proper regula-

tion of velocity of chlorinated water in filter filled with turnings of chemically pure magnesium, free chlorine can be removed, leaving only a negligible trace of hypochlorite. In practice, one has to deal with magnesium alloys; it is important to select material with which reaction (1) will predominate. Absolute freedom from carbide is essential: otherwise, acetylene will be formed and impart a disagreeable odor to the water. Electrolytic magnesium sometimes contains carbide, derived from graphite electrodes. Chlorine-magnesium reaction products are highly soluble in water, so that metal surface does not become obstructed. Hardness increase is negligible. Laboratory experiments described in detail.—*R. E. Noble.*

The Trend of Modern Taste and Odor Control. F. E. STUART. Water Works and Sewerage, 81: 9, 327-328, September 1934. Good modern practice may include heavy application of chlorine in pretreatment; addition of from 5 to 10 pounds per mg. of Aqua Nuchar in mixing chamber, to remove tastes, to improve coagulation, to stabilize sludge, and, subsequently in filter, to dechlorinate; and addition of ammonia and chlorine after filtration in sufficient quantities to meet local conditions. Should tastes encountered prove unamenable to above treatment, additional Aqua Nuchar may be applied as suspension directly to filter influent at average dosage of about 2 p.p.m. Addition of carbon particles to wash water, to "sweeten" the filters and increase their efficiency, is giving promise of becoming a worth-while procedure.—*R. E. Noble.*

A Thirteen Year Record of the Operation of a Municipal Water Department. L. R. BURCH. Water Works and Sewerage, 82: 7, 233-236, July, 1935. Discusses public water system of city of Tucson, Arizona, including operating records, number of consumers, office expense, pipe line maintenance, pumping costs, total operating costs, yearly improvements, total budgeted expenditures, bond interest and retirement fund, total costs and receipts, effect of metering, water rates, and revenues.—*R. E. Noble.*

Typhoid Case Traced to Polluted Stream. Municipal Sanitation, 6: 4, 117, April, 1935. Case developed in a small Connecticut city. State department's investigation of case history was fruitless. None of patient's family, friends, or neighbors, were carriers or ill of the disease. No unusual foods had been eaten, while water and milk supplies were entirely satisfactory. Further investigation, however, revealed an automobile accident in which patient had been catapulted into stream adjacent to highway and had evidently swallowed water therefrom. Stream was found to be polluted with domestic sewage.—*R. E. Noble.*

Discharge by Municipality of Raw Sewage into Stream from Which Another Municipality takes its Water Supply. Public Health Reports, 50: 27, 897-898, July 5, 1935. (North Carolina Supreme Court; Town of Smithfield et al. v. City of Raleigh et al., 178 S. E. 114; decided Jan. 28, 1935.) City of Raleigh discharged untreated sewage into two creeks at points approximately 33 miles

from town of Smithfield. Sewage flowed through creeks into Neuse River, source of Smithfield water supply. Section 7125 of Consolidated Statutes provided that any violator of statutes was guilty of misdemeanor. Statutes cover such cases as above, and plaintiffs asked that defendants be immediately enjoined from discharging untreated sewage into said creeks. Trial court denied petition and dismissed action, but provided that this judgment shall not be interpreted as an estoppel against plaintiffs, in case another action is brought for same cause, whenever it shall be made to appear that defendants are in a position to comply with the statute forming basis of this action. Supreme court upheld this decision.—*R. E. Noble.*

The Responsibility of Municipal and Privately Owned Water Plants for Supplying Infected Water. EDWARD R. STAPLEY. Southwest Water Works Jour., 17: 4, 9, 1935. Summary of some court decisions.—*O. M. Smith.*

The Relation of Activated Carbon to Water Purification. C. H. BILLINGS. Southwest Water Works Jour., 17: 4, 11–13, 1935. Results of tests on six activated carbons are given. Best carbon for water treatment is that which remains longest in suspension and shows high phenol removal capacity. Detailed methods of testing are given.—*O. M. Smith* (*Courtesy Chem. Abst.*).

Mottled Enamel. CRAWFORD A. MCMURRAY. Southwest Water Works Jour., 17: 4, 18–19, 1935. Ninety-eight of 400 children between 6 and 13 years of age developed mottled enamel by exclusive use for drinking purposes of water containing 4.4 p.p.m. of fluorine.—*O. M. Smith* (*Courtesy Chem. Abst.*).

Proposed Water Supply for Fort Smith, Arkansas. W. H. VAUGHN. Southwest Water Works Jour., 17: 6, 9–10, 1935. General description of legal obstacles, reservoir construction, and plant.—*O. M. Smith.*

Mottled Enamel in Texas. H. TRENDLEY DEAN, R. M. DIXON, and CHESTER COHEN. Public Health Reports, 50: 13, 424–442, March 29, 1935. **(A) The Panhandle and West Texas.** (1) This region constitutes largest mottled-enamel area in United States. As result of unusual population influx between 1920 and 1930, number of children affected has correspondingly increased. (2) Of 53 communities surveyed in 37 counties, only 6 could be classified as "negative" or "border line." (3) Fact that municipal water supplies of such large cities as Amarillo, Lubbock, and Plainview contain sufficient concentration of fluorine to produce among children a high percentage of mottled enamel, has developed an acute and urgent public health problem. **(B) East Central Texas.** (4) Endemic area of unknown size is reported in east central Texas between Austin and Dallas. (5) Of 13 communities surveyed, only 2 were classified as "border line" and none was classified as "negative."—*R. E. Noble.*

A Report on an Epidemic of Typhoid Fever in a Circus. K. E. MILLER, and H. E. MILLER. Public Health Reports, 50: 23, 778–797, June 7, 1935. (1) In July, circus employees suffered a diarrhea epidemic having winter cholera

characteristics. Two weeks later, explosive epidemic of 77 proved typhoid cases occurred. (2) Span of this epidemic, with exception of two cases, was confined to period July 22 to August 6. (3) Findings relative to typhoid carriers among food handlers are confusing, and their reliability in some instances is questionable. (4) Distribution of cases is more or less uniform throughout the circus personnel; all the larger groups, excepting trainmen, porters, and elephant men, were affected. The infection rate in the various groups presents no concentration that might be considered significant. (5) The possibilities (*a*) that infection was introduced through infected food or milk, or food which might have become contaminated in the process of preparation and serving, (*b*) that it was due to use of ice or to bathing in polluted water, and (*c*) that it was due to contaminated drinking water, have been duly considered. *Conclusions.* (1) The nature of the epidemic is such as to establish the hypothesis that infection was shared in common by practically all the circus groups, that it was received by all simultaneously and at one time only, that it came from without rather than from within the circus, and that it was a heavy dosage of contamination consisting of sewage organisms superimposed on typhoid infection. (2) The evidence is such as to make it highly improbable, if not impossible, for the epidemic to have been caused by infected food, typhoid carriers, infected ice, or bathing in polluted water. (3) Epidemic characteristics are in all respects typical of, and consistent with, water-borne infection. Fact that trainmen and porters, whose drinking-water supply is separate from that of the rest of the circus, had no cases of typhoid, tends to support this view. (4) While conclusion that epidemic had its origin in polluted drinking water appears reasonably certain, the exact place where the infection was picked up cannot be positively determined, though facts indicate that it was probably somewhere in western Pennsylvania.—*R. E. Noble.*

Million-Gallon Storage Tank Project Completed by Dallas is Second Elevated Tank of its Size Erected in Texas. JOHN E. WINDER. Water Works and Sewerage, 82: 4, 133–134, April, 1935. Article discusses seriousness of water shortage, plans for correction, some features of tank selected, and results obtained.—*R. E. Noble.*

Remote Reading Level Recorder. Engineering, 139: 361–2, April 5, 1935. Exhibit of Messers. George Kent Ltd. at Physical Society's Exhibition in England included a new instrument for distant recording, application of which to water level measurement is described. Apparatus is known as Kent V.D.I. telerecorder, the initials standing for "variable-duration-impulse," the principle employed. An electrical impulse is sent once every minute from the transmitter, located over the tank or reservoir, to the recorder, which may be situated, as for instance in an office, at any desired distance from the transmitter. Duration of the impulse varies from a minimum of 1½ seconds at zero reading to 57 seconds at maximum. Both transmitter and recorder are driven by small synchronous motors, supplied from frequency-controlled alternating current mains; but line current is supplied by accumulators. Transmitter and receiver can be linked by telephone lines. Instrument is so constructed that after any interruption of working, whether due to disconnection of the line wire, or to other cause, it will resume correct registration.—*R. H. Oppermann.*

Protective Coatings for Underground Pipes. A. B. DENSHAM, F. C. SMITH. Gas Journal, 211: 567-77, September 11, 1935. This Institution of Gas Engineers Report deals with urban conditions on the area of the Gas Light and Coke Company, of London, Eng. It briefly discusses relative corrodibility of wrought iron and steel and states that in the case of external corrosion the claim that wrought iron is preferable to steel does not hold. Greater part of the article is devoted to pipe coatings, methods of testing them, and discussion of their various types. A number of accelerated underground tests were made on bitumen, pitch, and cement coatings of various kinds, and it was found that both steel and wrought iron pipes corrode to much the same extent, when exposed to electrolytic corrosion. Order of merit of coatings investigated, judged from the electrical resistances, is roughly the same as order obtained when judged from extent of corrosion by an impressed current after 2½ years exposure, suggesting that electrical resistance test gives reliable measure of protective power of coating. Pitch troughing, bitumen-asbestos sheathing, bitumen-felt coating, bitumen-hessian coating, adhesive bandage, and insulation tape all give good protection. Appendices to the report treat of electrical resistance of pipe coatings, of details of methods of testing coatings, of methods of applying coatings, and of buying coated pipes, and include an extensive bibliography.—*R. H. Oppermann.*

Constant Pressure by Variable-Speed Pumps. I. L. BOYUM. Power, 79: 232, May, 1935. There are three methods in general use for maintaining approximately constant pressures in water supply systems, namely, pneumatic, or sealed, reservoirs, elevated tanks, and continuous running centrifugal pumps, discharging directly into mains and equipped with some means to govern discharge pressure in accordance with demand. For last-named type of installation, an all-electrical pressure regulation system is available, which includes variable-speed motor on the pump, a pilot-motor-operated rheostat in motor secondary circuit, a set of pilot-motor control contactors, and pressure regulators to position contactors. This control uses two separate pressure regulators, for low and for high pressure, respectively. Usual arrangement of this system is semi-automatic, in which operator pushes a start button to close the primary contactor, after which, pressure is regulated automatically. A fully automatic system can be used, in which operator starts the first pump, after which, additional pumps are connected as increase in load warrants. An outstanding feature of the equipment is the elimination of burning contacts and insurance of completion of each individual step of regulation.—*R. H. Oppermann.*

Celluloid is Used by Pipe Line to Combat Corrosion and Stress. L. E. BREDBERG. Oil and Gas Journal, 33: 43, 13, 32, March 14, 1935. Pipe line companies are bending every effort to solve the very costly corrosion problem. Though data obtained after a year's use of celluloid by a large western company cannot be taken as conclusive, yet it is believed that this material is very effective. The Du Pont Viscoloid Co. "cellulose nitrate plastic" known as "Pyralin" comes on market in strip form, 17 inches wide and $\frac{10}{1000}$ inch thick, and is shipped in rolls. It can also be obtained $\frac{20}{1000}$ inch thick. Strips are

cut in two for wrapping 6- and 8-inch pipe; 20-inch strip is available for very large pipe. Since Pyralin is inflammable, it is stored in a dark, well-ventilated warehouse; it flashes at 525°F.; its tensile strength is 8000 pounds per square inch; it is brittle at —20°F. and softens at 220°F., rapid decomposition beginning at 325°F. When softened, it will expand in volume 5 percent and when hardening, it will shrink in volume 12 percent; does not crack on shrinking, but behaves like stretched rubber. Atmospheric exposure does not cause it to deteriorate, but sunlight, which contains ultra-violet rays, will break it down. A softening compound (alcohol, ethyl acetate, and water) is used to render it sufficiently pliable to fit over collars and other projections. A pyralin quick-setting cement is applied by hand with a brush to edge of roll of wrapping, as it is spiralled around the pipe.—*R. H. Oppermann.*

Cathodic Protection of Pipe Lines from Soil Corrosion. SCOTT EWING. Gas Age Record, 75: 179-82, 219-22, 239-42, 261-4; March 2, 9, 16, 23, 1935. Detailed report of the American Gas Association Sub-Committee on Pipe Coatings and Corrosion in form of a treatise, describing principles of cathodic protection and their practical application. Cathodic protection is in many cases a reliable means for effectively reducing, or even entirely overcoming corrosion of underground pipe lines. Whether it is economical to use it on any given system, will depend upon its cost and the savings which will be effected over other protective methods. Its cost will depend primarily upon the condition of the coating on the pipe line. If the line is bare, or is an old one on which paints, or other inadequate coatings, were used, the cost of cathodic protection would be large. Whether cathodic protection would be economical in such cases will depend upon many considerations and cannot be answered in general. Some engineers believe that it may be feasible to use cathodic protection on some bare lines. However, it will probably be extended to many lines where its use is now considered impossible, as more experience and skill are acquired in the design and operation of cathodic protection circuits. Any new line can be built so that cathodic protection may be applied to it, with very little expense, at any future time. The most extensive use so far, to which it has been put, is on lines that have begun to develop leaks, but whose coatings show reasonably high resistance. Report describes methods for making preliminary surveys for estimating costs and for determining where electrical drainage stations should be located. Methods are also given for determining whether, or not, the protection is effective, at any time after the installation is in operation.—*R. H. Oppermann.*

Water Supply and Purification Developments During 1934. I. W. MENDELSOHN. Water Works and Sewerage, 82: 1, 1-7, January, 1935. Review describing new developments and trends.—*H. E. Hudson, Jr.*

Experiences in Freezing and Thawing of Water Mains and Services. ALBERT A. ROSS. Water Works and Sewerage, 82: 1, 8, January, 1935. At Lexington, Mass., during severe winter of 1933-34, 5 percent of services, 348 meters, and 6- and 8-inch mains were frozen. Thawing was effected either electrically, or with steam: technique is described. Acquisition of welding unit is recom-

mended. Prescribed service pipe depth was increased to 6 feet in 1934. Frozen meters were removed and replaced with nipples. Frozen hydrants were thawed with calcium carbide. Copper services and a large main with comparatively new compound joints required steam thawing.—*H. E. Hudson, Jr.*

Practical Aspects of Storing and Handling Ferric Chloride. J. M. POTTER. Water Works and Sewerage, **82**: 1, 12, January, 1935. Ferric chloride is commercially available in 3 forms: liquor, crystals, and anhydrous. Liquor is a dark brown aqueous solution of oily appearance varying in strength between 39 and 45 per cent of $FeCl_3$. Concentration depends upon the season, being as high as danger of crystallization will permit. Graph, showing relation between solubility and temperature is given. Liquor is shipped in rubber-lined tank cars, whence it is removed either by siphoning, or by air ejection. Rubber-lined storage tanks are preferred. Crystalline ferric chloride ($Fe_2Cl_6 \cdot 12 H_2O$) contains 60 percent $FeCl_3$, comes in lump form, and is slowly soluble. Material is marketed in barrels in which it may be stored. Anhydrous $FeCl_3$ is readily soluble. It is shipped in steel drums which may be stacked for storage. Anhydrous $FeCl_3$ is shipped most cheaply and requires least storage space. Hygroscopic nature of $FeCl_3$ forbids dry feeding and, due to corrosiveness of its solutions, special equipment is advocated. Suggested dissolving procedure is given. For application to water, 2 to 4 percent solutions are best. Reinforced concrete painted with $FeCl_3$-resistant paint is suitable for solution tanks. Positive displacement, or variable siphon head, feeders are recommended. There are 7 illustrations.—*H. E. Hudson, Jr.*

A Study of Filtering Materials for Rapid Sand Filters. Part 4. Washing Rapid Sand Filters. J. R. BAYLIS. Water Works and Sewerage, **82**: 1, 20–25, January, 1935. Brief history of filter washing methods is given. This, combined with author's extensive experiments, shows that best backwashing is obtained with 50 percent sand expansion; but since this does not keep sand clean, other force is necessary. Formation and growth of mud balls is described, and detailed accounts of mud ball behavior in filters containing various materials are given. Forces and laws governing action of mud balls in filters are discussed. Tests with natural and artificially prepared mud balls showed that sand expansions of 50 percent help to prevent formation of clogged masses by scattering mud balls, but that there is no rate of backwash which will keep a filter in the desired perfect condition. Surface wash is highly recommended and a suggested design is set forth. With surface wash, filters can be kept clean with from 12 to 20 gallons per minute per square foot of total wash, without loss of sand.—*H. E. Hudson, Jr.*

Old Standpipe Rehabilitated by Guniting: $4,000 Spent, $14,000 Saved. V. VON GEMMINGEN. Water Works and Sewerage, **82**: 1, 30, January, 1935. Guniting inside and outside of old iron standpipe rendered it watertight and sightly and saved otherwise imperative construction of new one.—*H. E. Hudson, Jr.*

Are Water Utilities a Good Investment? CHARLES H. CAPEN, JR. Water Works and Sewerage, **82**: 2, 42, February, 1935. Data from the water works of

New Jersey are analyzed. Author concludes that water utilities are safer than most business ventures. Because they charge relatively higher rates, large supplies prove more successful than smaller ones. Rates charged by small companies generally appear inadequate. Municipal systems too often do not charge enough to prevent deficits. A 20-year graphical comparison of average deficits, net income, and revenue is presented.—*H. E. Hudson, Jr.*

The Elimination of Check Valve Slam. H. K. PALMER. Water Works and Sewerage, 82: 2, 45, February, 1935. Mathematical criteria are explained and air chambers for surge absorption are described and recommended.—*H. E. Hudson, Jr.*

Keeping Lime Solution Lines Clean. L. H. ENSLOW. Water Works and Sewerage, 82: 2, 57, February, 1935. At the Richmond, Va., filtration plant, clock-regulated flushing and occasional chlorination are helpful. Rubber hose, rubber-jointed, is preferred for feed lines.—*H. E. Hudson, Jr.*

Journal of the New England Water Works Association, Volume 49, No. 1, March 1935. Experimental Study of Porous Plates for Use in Filter Bottoms for Rapid Sand Filters. THOMAS R. CAMP. 1–13. Hydraulic design of underdrains is determined almost wholly by back-washing function: i.e., so to distribute wash water to under side of bed, that each unit area receives same amount, that whole bed is evenly washed, and that friction loss is evenly apportioned. Friction loss usually consists of head losses through manifold laterals, orifices, gravel, and filter medium. Because of difficulty of estimating accurately variation in available head from orifice to orifice, it is customary to make head loss through orifices large, in comparison with this variation, so as to obtain comparatively uniform flow through all orifices. In practice, absolutely uniform discharge from each and every orifice is unnecessary, since gravel layer effectively provides for distribution of water. Work of GILBERT and others has demonstrated that loss of head in gravel layer is concentrated in finer material just under sand. Gravel layer has 3 functions: to support filtering material, to dissipate high velocities of jets, and to distribute wash water uniformly to filter bed. Orifice system may be dispensed with in favor of equalizing chamber under gravel. Failure of gravel to stay in place has led to efforts to dispense with gravel also. A system of porous plates, for example, set above equalizing chamber, will not be displaced during washing. Porous plates, moreover, possess this advantage over other orifice systems, that rate of wash may be increased without undue increase in head loss. Doubling rate of wash corresponding to head loss of 4 feet will result in head loss of only 8 feet in case of porous plates, while 16 feet will be required with orifice systems. Flow through porous plates at normal velocities is stream-lined and viscous. Studies are now under way to determine extent of clogging to which porous plates are subject and most efficient method of cleaning them. **Gain in Head at Take-Offs.** W. E. HOWLAND. 14–21. Measuring total head on either side of leak in flowing pipe, author found apparent gain in head on down-stream side when ratio of side flow to main flow is less than 1 to 2. Leak, or branch, outlet may scoop off relatively slow moving edge layers of water, leaving fast-

moving, and therefore high energy containing, central core of water to flow past take-off. Where number of side branches is considerable, cumulative gain in head may be sufficient to call for consideration in computation of pressure distribution along pipe. Inadequacy of conventional energy equation to represent analytically with sufficient accuracy effects of side outlets has led to development of new energy equation which fits the facts more closely. Velocity head is ascribed to water at upstream section about to leave through take-off, which is smaller than average velocity head at same point. Ratio, C, of these velocity heads is of prime importance in mathematical explanation of phenomenon. **Experiences with the Public Works Administration and Other Governmental Agencies for Public Works Construction.** A Symposium. 22-52. P. W. A. state engineer, 3 representatives of state health departments, and one engineer in private practice review projects in New England and New York. In following discussion, HAROLD J. LOCKWOOD of P. W. A. interprets rules and regulations which have caused most dissatisfaction. **Improvement of the Water Supply of Braintree, Mass.** ROBERT SPURR WESTON. 53-65. Inferior quality of water supplied from Great Pond to Braintree's 18,000 inhabitants has caused many complaints. Water is highly colored, possesses an objectionable taste and odor, and is gathered from highly-polluted, unsewered catchment area. On May 16, 1934, new 2.5-m.g.d. rapid sand filtration plant was put into operation. Both alum and sodium aluminate were applied as coagulants, together with activated carbon to remove taste and odor, and after filtration, a small chlorine dosage. Recently, aluminate has been eliminated and carbon dosage diminished. Treatment has changed disagreeable water to one of high quality. At relatively low cost, plant capacity may be increased to 3.33 m.g.d. **Status of Cross-Connection Regulations.** C. W. MOWRY. 66-71. Policies of State Health Authorities (1934) were as follows:—12 States had regulations prohibiting cross-connections: 11 States required that all cross-connections be eliminated unless protected with approved type double check-valves (of whom, 4 prohibit new cross-connections); and of remaining 25 States having no regulations, 5 have adopted policy of prohibiting cross-connections, policy of 5 of far western States is unknown, and 15 remaining States judge each case on its merits eliminating cross-connections where possible and permitting certain existing cross-connections to remain, if protected by double check-valves. Elimination and safe-guarding of cross-connections has been given impetus by instructions from Surgeon General of U.S.P.H.S. to State health officers that full certification may be given water supplies only if cross-connections are eliminated, or equipped with modern protective devices. In most cases, local water and health authorities carry out details of enforcing State department regulations. Experience has demonstrated that vast majority of cross-connections are unnecessary and can be readily eliminated. For few cases where immediate elimination is impracticable, use of all-bronze rubber-seated double check-valves, regularly tested and cleaned, has thus far provided reliable safeguard. Automatic chlorinators for disinfecting secondary waters for fire protection, etc., are valuable where machines are properly maintained. **Activities of the New England Regional Planning Commission.** VICTOR M. CUTTER. 72-75. Immediate objectives of commission are:—(1) ten year program of public

works, including parks, parkways, highways, and other developments; (2) study of transportation facilities, with plans for coördination; (3) report on major existing and desirable land uses; and (4) report on suggested legislation. Of particular interest is the Connecticut River, its pollution, navigation, recreation facilities, etc. Example of what may be accomplished is splendid result achieved by foresters of New Hampshire in protecting beauty of wooded areas. **The Water Works of Saint John, New Brunswick.** Hon. R. W. WIGMORE. 76–81. With population of but 65,000, city has fresh water supply from two distinct lake systems, sufficient to serve city of size of Boston. Development is traced from early days. Cholera epidemic of 1854 stirred inhabitants to realization of necessity for plentiful water supply. Present system, valued at 4 million dollars, supplies 7½ b.g. of water annually. Income from meters, $59,000; from water assessment, $207,000. Private dwellings are not metered but are assessed annually as follows:—0.25 percent of valuation, plus $4 for each family in house, $4 for each toilet, $3 per bath, and 50 cents per wash basin. Stores are charged 0.25 percent on stock. Meter rates vary from ½ to 2½ cents per hundred gallons. **Sanitary Scoring of Surface Water Supplies.** A. D. Weston. 82–103. Primary object of scoring system is to improve inferior water supplies, and to maintain uniformly high standard in others by providing balance sheet whereby sanitary assets and liabilities may be compared. Although its value is unquestioned, scoring system proposed by CHASE and committee has failed to come into popular use, because its proper application has not been properly understood. Provision has been made for scoring surface water and ground water supplies. Each group in turn is divided into 3 parts: pollution hazards, protective measures, and final quality of water supplied. (1) *Pollution Hazards.* Total credit for no hazard, 60. Penalties are charged for population contributing sewage directly, or indirectly, to watershed, for animal pollution, highways, railways, industrial wastes, lumber camps, bathing, and picnicing. Cross-connection to polluted supplies invokes a penalty of 25 to 50, unless double check-valves are provided, which reduce penalty to 5, with further reduction to 3 for fire-pump chlorinators. (2) *Protective Measures.* Credit given for proper sanitary regulation, 100 percent ownership of watershed, adequate storage time, filtration, chlorination, competent operators, and laboratory supervision. Total credit of 30 allowed, but double filtration and double chlorination merit 10 points additional, while political interference warrants penalty of 15. (3) *Quality Conditions.* Total credit, 30. Turbidity, color, odor, taste, hardness, corrosiveness, presence of iron and manganese, and bacterial quality are considered. Penalty for more than 1 p.p.m. of iron and manganese is 3; for objectionable taste and odor, up to 10 points. *Summary.* Total possible score for 3 parts is 120, but 100 is considered perfect; 75–100, good; and 50–75, fair. Of 10 municipalities scored, results ranged from 1 to 82. Value of scoring system is that it eliminates subjective element in scientific appraisals, and stimulates popular interest in municipal activities. Laymen understand the "old score card." **Soil Corrosion Report.** H. W. CLARK. 110–112. Data from Bureau of Standards Research Papers Nos. 329 and 638 indicate that character of soil controls rate of corrosion; in same soil all common ferrous materials corrode at same rate. High-silicon cast iron loses practically no

weight by corrosion and shows little evidence of pitting, although in general non-ferrous materials and alloys, zinc, copper, and lead, are more resistant. While reports are inconclusive, corrosion in any one locality may apparently vary remarkably from year to year, due to variations in temperature, in weather, in soil, and in specimens themselves. Zinc coating weighing 1 ounce per square foot of exposed surface should extend life 6 years is very corrosive soil and much longer under more favorable conditions.—*T. F. Donahue.*

NEW BOOKS

Papers, Joint Program, Fifth Arkansas Water and Sewage Conference and Tenth Oklahoma Water and Sewage Conference. Fayetteville, Ark., April 16–18, 1935. 11 x 81 inches, mimeo. General Extension Service, University of Arkansas, Fayetteville, Ark. **Proposed Fort Smith Water Supply.** W. H. VAUGHN. 6–8. Historical notes on former supply and description of proposed and adopted plans for new supply, including impounding dam, 6-m.g.d. filter plant, and 25-mile gravity supply line to city. All pumping will be eliminated eventually. Total cost of project, financed by PWA, estimated at $1,580,000, including dam and roads, $390,000; filter plant, $175,000; pipe line, $858,000; and other overhead items. **Public Utility Service and Discrimination.** L. M. REBSAMEN. 8–17. Digest of public utility commission rulings and court decisions relating to obligation to render service, public relations, inability to serve, service duties, service to competitors, excuses for failure or refusal to serve, ownership of connections, instruments, and equipment, maintenance of meters, fire protection service, water pressure, quality of water, and discriminatory practices. **The Responsibility of Municipal and Privately Owned Water Plants for Supplying Infected Water.** E. R. STAPLEY. 17–20. City providing Water supply service acts in a private, or corporate, capacity. Plaintiff claiming damage must prove that supply furnished was polluted and that there was negligence in operation of plant. Plaintiff must also prove that disease was contracted from supply in question and not from other sources. Plaintiff must not be guilty of contributory negligence. City providing Water supply implies warranty that water is fit for human consumption unless proper notification to the contrary is made. **Ground Water in Arkansas.** G. C. BRANNER. 20–24. Ground waters in highland sections of state, with exception of those obtained from river bottom sands and gravels, will probably be of little importance for municipal or industrial supply. Those in lowland, or Gulf Coastal, section have been, and will continue to be, important for municipal and industrial supply and irrigation. **Contamination of Well Water Supplies.** E. W. BOYCE. 24–31. Discussion of sanitary requirements for well location and construction, and of proposed regulations of Kansas State Board of Health. **Recent Developments in Water Treatment.** E. W. BOYCE. 31–34. Discussions of modern progress in water treatment considered as a manufacturing process. Taste and odor control, softening, and disinfection are fields in which recent advances have been rapid. **Bacteriological Tests.** A. B. JEWELL. 34–38. Discussion of the proper popular interpretation of tests and their significance under various conditions. **Developments of Sewage Treatment and Equipment for Sewage Treatment.** L. H.

SCOTT. 38–44. Description of North Side sewage treatment plant of Oklahoma City, with special emphasis on the manufacture of ferric chloride.—*R. L. McNamee.*

Report on Water Pollution. The Special Advisory Committee on Water Pollution of the National Resources Committee. July, 1935. 82 pp. Washington, D. C.

This report is a comprehensive resume of fundamental information on stream pollution, and includes the status of water pollution laws, of water pollution from the standpoint of public health, of water pollution from biological considerations other than those of public health, of industrial waste treatment, and of standards of water quality. The data assembled, while indicating definite progress in stream pollution control, are nevertheless in many ways a serious indictment of the present situation in the country. With the view of bringing about a more rapid improvement, the Committee recommends the following policy in relation to water-pollution control:

"1. That where drainage area authorities exist for the comprehensive development and control of water, their scope be made sufficiently broad to include control of pollution.

"2. That no basic changes in existing Federal law with reference to water-pollution control be made until the experimental program presented later herein shall have indicated whether or not such changes are desirable and feasible.

"3. That simplification and coördination of State laws be effected to provide for the following minimum requirements:

(a) Adequate administrative control.

(b) Delegation to the administrative agency of power to determine the nature and extent of pollution prohibited by the statutes, and to establish limitations of pollution.

(c) Appropriate mandatory powers, particularly the power to compel the installation of essential remedial works and force other necessary action.

(d) No limitation on the taxing or bonding power of municipalities, when applied to remedial works ordered by the administrative agency.

(e) Power to require facilities through sewerage districts, sanitary districts, or otherwise, that will enable municipalities and industries to comply with the law.

"4. That broader authorization for research be granted to those agencies of the Federal government which are already concerned with various phases of the problem, and that adequate funds be provided for properly coördinated water-pollution investigations.

"5. That powers and funds be granted to an appropriate Federal agency to institute a coöperative program of investigation with legally constituted State agencies for such special studies as appear desirable, and particularly for the development of appropriate standards for water use and control.

"6. That in order to stimulate the construction of pollution-abatement works, funds for the purpose be made available by the Federal government to local public and private agencies on a grant-in-aid or loan basis. Lacking

any precedent for the designation of appropriate bases for such allocations, the Committee recommends the creation of a demonstration unit on a river system selected for that purpose."

It would seem pertinent to urge that the research program recommended by the Committee include a thorough investigation of the actual effects of various kinds of pollution on lakes and streams since, as the report points out, there is a dearth of knowledge on this phase of the problem. Such basic information would be extremely helpful in developing further the standards of water quality, and in evaluating the actual damage caused by pollution, all of which would be very useful in obtaining the correction of unsatisfactory conditions on a safe and equitable basis. Anyone interested in stream pollution problems will find much valuable information in this report.—*H. A. Whittaker.*